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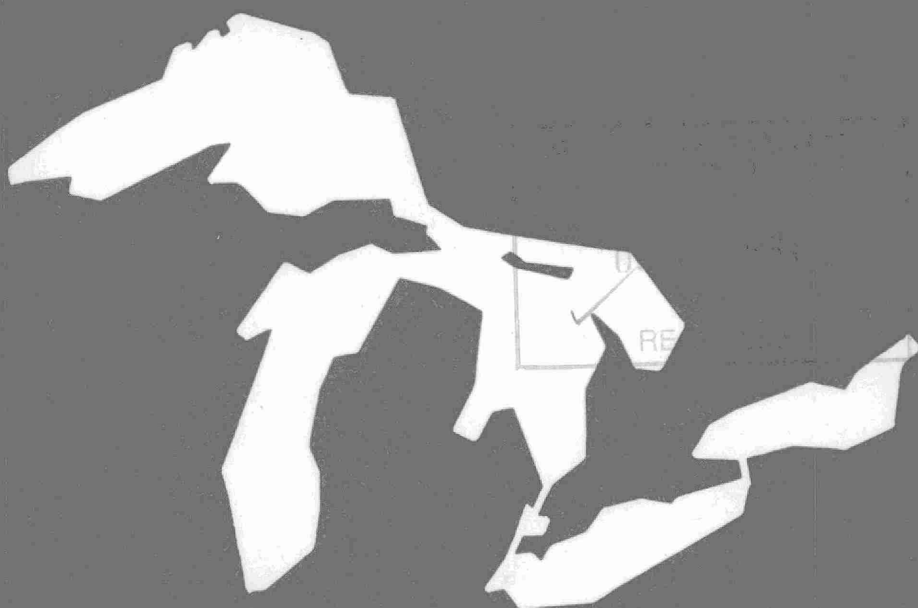
Ministry of the Environment
Ontario

Manual of Practice for Urban Drainage

Research Report No. 104



Research Program for the Abatement of Municipal Pollution under Provisions of the Canada-Ontario Agreement on Great Lakes Water Quality

CANADA-ONTARIO AGREEMENT

RESEARCH REPORTS

These RESEARCH REPORTS describe the results of investigations funded under the Research Program for the Abatement of Municipal Pollution within the provisions of the Canada-Ontario Agreement on Great Lakes Water Quality. They provide a central source of information on the studies being carried out in this program through in-house projects by both Environment Canada and the Ontario Ministry of Environment, and contracts with municipalities, research institutions and industrial organizations.

Inquiries pertaining to the Canada-Ontario Agreement RESEARCH PROGRAM should be directed to -

Wastewater Technology Centre
Canada Centre for Inland Waters
Environment Canada
P.O. Box 5050
Burlington, Ontario L7R 4A6

Ontario Ministry of Environment
Pollution Control Branch
135 St. Clair Avenue West
Toronto, Ontario M4V 1P5

MANUAL OF PRACTICE ON URBAN DRAINAGE

RESEARCH PROGRAM FOR THE ABATEMENT
OF MUNICIPAL POLLUTION WITHIN THE
PROVISIONS OF THE CANADA-ONTARIO
AGREEMENT ON GREAT LAKES WATER QUALITY

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FOREWORD

This manual is intended to lay a foundation for the introduction in Ontario of policies related to the prevention and abatement of urban drainage problems. State-of-the-art concepts, analytical methods and technology related to the solution of urban drainage problems are presented for the information of consultants, planners, developers, and municipal officials.

It is expected that the usefulness of this manual will not be restricted to Ontario. With appropriate modifications it can provide a background for urban drainage policies that may be developed by the governments of Canada and the other provinces.

This manual presents planning and design concepts, analytical and design methodologies, and technological alternatives for prevention and abatement of problems related to the quantity and quality of storm runoff from developed urban areas and areas undergoing urban development. Solutions to these problems are identified and depend on planning, design, and operation of sanitary, storm, and combined sewage systems. This manual is not a "design manual" containing the information needed to plan and design urban drainage systems. Rather, it is intended as a presentation of the concepts which can be used to derive solutions to the problems. Full advantage has been taken of the opportunity to refer to many excellent references that deal in detail with topics discussed herein. The manual consolidates the research and development findings of the Urban Drainage Subcommittee program of the Canada-Ontario Agreement as well as information from other Canadian programs and work conducted in other countries, especially the United States, related to storm and combined sewage control.

This manual addresses the following specific objectives:

i) To identify the seriousness of urban drainage problems. Pollution caused by discharges of municipal and industrial sewage effluents is well recognized. Other urban drainage problems have not received the same degree of recognition.

ii) To identify and describe new effective solutions to urban drainage problems. Recent advances in research, development, and planning and design practices in Canada and elsewhere have yielded new concepts and improved technology and methodology for the prevention or solution of urban drainage problems. This manual is intended to describe alternative solutions to urban drainage problems and to present methodologies for selecting alternative solutions.

iii) To present urban drainage problems and solutions in a framework of inter-agency cooperation. Urban drainage problems are the concern of many government agencies and, in some circumstances, pursuit of the objectives of one agency may create conflict with the objectives of other agencies. This manual is an attempt to develop a consistent and coherent approach to the identification and solution of urban drainage problems in Ontario, based on the consensus of a working committee that included representatives from municipal, provincial, and federal government departments. It is intended to rationalize the concepts of all these agencies into an approach that examines the whole system within which urban drainage problems arise and are solved, and to outline administrative procedures for implementing solutions.

iv) To achieve urban storm water management. If storm water is properly controlled and used, the potentially negative effects of urban development can be minimized, and maximum advantage can be taken of opportunities to use this resource to improve urban environments.

v) To contribute to the achievement of the general and specific water quality objectives for the Great Lakes that have been agreed upon by the governments of Ontario and Canada.

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CREDITS AND ACKNOWLEDGEMENTS

The manual has been developed through the cooperative effort of a working committee with membership as follows:

J.J. Armstrong	Ministry of Transportation and Communications
D.B. Cane	Ministry of the Environment
R.E. Crawford	Municipal Engineers Association of Ontario
L.A. Drennan	Environment Canada
T. Kurtz	Ministry of Natural Resources
S. Llewellyn	Environment Canada
G.O. MacLellan	Ministry of Housing
R.A. Madill	Ministry of Transportation and Communications
J. Marsalek	Environment Canada
F.S. Martin	Ministry of Housing
Z. Novak	Ministry of the Environment
D. Persaud	Ministry of the Environment
G. Sardesai	Ministry of Natural Resources
P. Seto	Ministry of the Environment (Co-Chairman)
E.R. Simonen	Municipal Engineers Association of Ontario
F. A. Tonelli	Ministry of the Environment
H. C. Torno	United States Environmental Protection Agency
D.H. Waller	Nova Scotia Technical College
D.B. Walter	Environment Canada
B.R. Ward	Ministry of the Environment
D.G. Weatherbe	Ministry of the Environment (Co-Chairman)

The six chapters of the manual were compiled under the supervision of the following principal chapter writers:

Chapter 1	Urban Drainage Problems,	D.H. Waller
Chapter 2	Urban Drainage Planning,	D.G. Weatherbe
Chapter 3	Characterization of Flows in Drainage Systems,	J. Marsalek
Chapter 4	Source Controls for Quantity and Quality of Urban Drainage,	F.A. Tonelli
Chapter 5	Collection, Storage and Treatment	F.A. Tonelli
Chapter 6	Administration and Implementation of Urban Drainage Solutions,	B.R. Ward

Numerous projects have been carried out by external contractors and government researchers for the Urban Drainage Subcommittee and are referred to in this manual. The manual committee would like to thank the many agencies, consultants and individuals who provided input material for the manual. The efforts of James F. MacLaren Limited, Paul Theil and Associates Limited, Gore and Storrie Limited, DeLeuw Cather Limited, and IEC International Consultants are acknowledged.

The manual is a result of the efforts of the working committee and does not necessarily constitute the policy of the governments or agencies the members represent.

1 URBAN DRAINAGE PROBLEMS

1.1 Introduction

Seventy percent of the population of Canada and more than eighty percent of the population of Ontario live in urban areas. One result of the continuing urbanization process is that drainage problems, in newly developed areas and in existing urban areas, are intensified. The Foreword indicated briefly the scope of urban drainage problems that are considered in this manual; this chapter is intended to describe the nature and severity of these problems and to serve as a prelude to following chapters that deal with planning and implementation of solutions to these problems.

Table 1 summarizes urban drainage problems in developed and developing areas. Figure 1, showing sources and movements of water and pollutants in urban systems, may help to illustrate some of these problems. Table 1, and the discussion that follows, do not include problems associated with municipal sanitary sewage and industrial waste discharges; these problems are well recognized and information about abatement technology is available elsewhere.

Where possible, points made in the chapter have been illustrated by examples of drainage problems in Ontario Communities. Unless otherwise indicated, the information has been taken from two recent reports [1,2].

1.2 Problems in Developed Urban Areas

1.2.1 Pollution from combined sewer overflows

The sewerage systems of older Canadian cities originated as combined systems -- conveying both sewage and surface runoff. Development of these systems was one of the earliest effects of urbanization. In built-up areas, local pollution resulted from the introduction of water-carried sewerage systems and the inability of soil disposal systems to handle increasing quantities of sewage. Stream channels, which had been enclosed to serve as storm sewers, formed the basis of combined sewerage systems when sewer connections from houses and businesses were permitted or required. As these communities developed, combined systems were extended. When interceptors and treatment plants were installed, provision was made for wet weather overflows of mixed sewage and storm

TABLE 1. URBAN DRAINAGE PROBLEMS

Problems	Causes	Effects
	Developed Areas	
Combined Sewer Overflows	- conventional interceptors and treatment plants handle only two to four times dry weather flow	- pollution of receiving water by wet weather overflows of mixed sewage and surface runoff
Urban Surface Runoff Pollution	- build-up of pollutants on impervious areas from all urban activities	- pollution of receiving water by nutrients, bacteria, sediment, metals
Overloaded Sewerage Systems	- infiltration/inflow - redevelopment to higher density land uses - development of upstream lands	- local flooding - foundation damage - health hazards associated with sewage backup - pollution of receiving waters by wet and dry weather by-passing of sanitary sewage
	Developing areas	
Increased Volume and Rate of Surface Runoff; Lowered Groundwater Tables	- removal of protective surface vegetation and paving during construction - drainage systems installed to deal with local flooding - construction in flood plains, and filling of ponds and swamps	- local flooding - erosion, and siltation of receiving waters - reduced base flow in streams - loss of natural storage areas and of recreational and aesthetic amenities
Construction Activities	- top soil removal - heavy machinery disturbs stream banks - excavation and grading	- turbidity and sediment in receiving waters

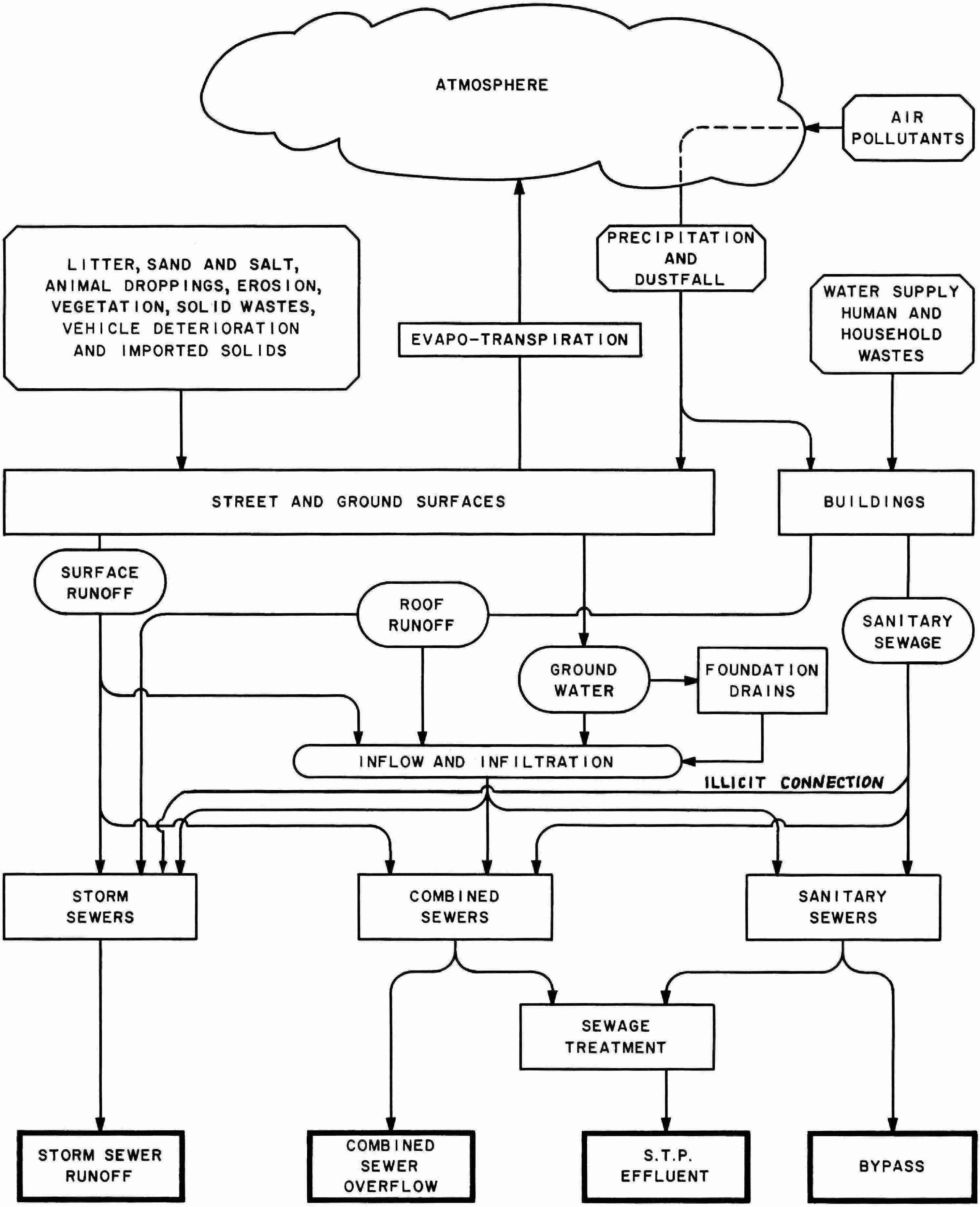


FIGURE 1. SOURCES AND MOVEMENTS OF WATER AND POTENTIAL POLLUTANTS IN URBAN DRAINAGE SYSTEMS

water that could, in major storms, be one hundred or more times dry weather flows. Interceptor and treatment capacities were usually designed to spill flows in excess of two to four times dry weather flow; larger capacities increase costs and achieve only limited reductions in frequency or volume of overflow in many cases.

Most approving authorities today require the installation of separated sewers -- using parallel pipes for sanitary sewage and surface runoff -- in new systems or when existing systems are extended. These requirements are based on concern about pollution caused by combined sewer overflows. In some areas separated systems are more economical because cost savings are realized by installing only sanitary sewers and avoiding or deferring storm drainage costs by allowing surface water to flow in ditches and natural channels.

About one-half of the urban population of Canada, and nearly 40 percent of the urban population of Ontario, are served by combined systems [3,4]. These estimates are low because they do not include systems that are effectively combined because of inter-connections between separated and combined systems.

The nature of pollution from combined sewer overflows is indicated by Table 2, which compares the composition of combined sewage with that of treated effluents and surface runoff. Except for total nitrogen, mean pollutant concentrations in combined sewage are higher than those in the other sources. The effects of combined sewage are more pronounced than this Table might suggest because (a) scour of deposited solids yields concentrations at the beginning of storms that greatly exceed the mean values shown in the Table, and (b) the effects of combined overflows are pronounced because large volumes of wastewater are discharged during short time periods. Discharges from combined sewer overflows in wet weather may greatly exceed wet weather loads from treatment plants, and can equal or exceed the total annual load discharged from a secondary treatment plant. As dry-weather treatment objectives have been approached or attained, the effects of combined sewer overflows and the need to control pollution from this source have become more evident. Many of the approaches that have been proposed or adopted have recognized that the problem is not the combined sewerage system *per se*, but pollution caused

TABLE 2. COMPOSITION OF SEWAGE AND STORM WATER

	BOD (mg/L)	SS (mg/L)	Total N (mg/L)	Total P[f] (mg/L)	Coliform[c,d] per 100 mL	Fecal Coliform[c,d] per 100 mL
Raw Sewage[a]	165	225	30	6.5	10^8	10^7
Treated Sewage[a]						
- Primary[e]	50	50	22	1.0	10^7	10^6
- Secondary	17	23	18	1.0	10^4	10^3
Combined Sewer Overflow[b]	41	190	8.3	1.4	10^7	10^6
Surface Runoff[b]	14	170	3.5	0.35	2×10^4	5×10^3

Notes: [a] measured flow-weighted mean

[b] calculated flow-weighted mean for Ontario Great Lakes communities [5]

[c] These values are representative of values recorded in Canadian and U.S. communities.

[d] Ontario Total Body Contact Recreational Standards:

Total Coliform 10^3 per 100 mL.
Fecal Coliform 10^2 per 100 mL.

[e] Primary effluent concentrations are lower, compared to the values from raw sewage, than would be expected for conventional primary treatment. Raw sewage concentrations are based on data from all plants in Ontario, while primary effluent concentrations are based on data from primary plants only, which in Ontario receive generally weaker sewage.

[f] Treated sewage total phosphorus concentrations are based on the assumption that all treatment plant effluents meet the International Joint Commission target of 1.0 mg/L.

by overflows, and that methods of reducing the amount and frequency of overflows, or of improving the quality of the wastewater that is overflowed, are alternatives to the often-proposed but seldom-adopted practice of combined sewer separation.

The effects of combined sewage overflows include aesthetically offensive floating solids and sediment, turbidity and reduced dissolved oxygen concentrations that can produce fish kills, and bacteriological effects on water supply sources and recreational areas. These problems have been recognized in Ottawa, Black Creek in the Borough of York, and the City of St. Thomas. In St. Thomas overflows of untreated domestic and industrial wastes have been blamed for extensive pollution of neighbouring streams and creeks [6]. Bacteriological quality of bathing beach waters at Cornwall improved during a dry summer when combined sewer overflows were at a minimum [7]. Section 1.4 reviews two studies of the effects of combined sewer overflows on the Thames River basin and on the Great Lakes.

1.2.2 Pollution from surface runoff

Increasing awareness of the potential magnitude of wet weather pollution has focused attention on the quality of urban surface runoff. The sources of this pollution are illustrated in Figure 1.

Table 2 compares pollution concentrations and loadings from surface runoff with those in sanitary and combined sewage. Coliform concentrations, although low in comparison with values in treated sewage and combined overflow, are high enough to exceed recreational water bacteriological standards. Deterioration of the bacteriological quality of the Rideau River has been attributed to surface runoff following urbanization.

BOD and nutrient concentrations in surface runoff are lower than those in the other two sources. Suspended solids concentrations approach those in raw and combined sewage, and much higher values have been recorded in areas where construction activity occurred.

Although nutrient concentrations in runoff are low, annual nutrient loads may be significant when urban runoff reaches an urban lake. Midland Park Lake, a small (146 ha) recreational lake in the town of Midland, Simcoe County, is an example of a small urban lake exposed to the threat of

development in its drainage basin. A report by the Ontario Ministry of Environment [8] suggests that if development in the drainage basin is to continue, untreated storm water should not be discharged to the lake. Pollution by phosphorus and bacteria was of particular concern and potential damage was expected from construction-related sedimentation.

Surface runoff is also the means by which most of the salt used for ice control is eventually removed from streets. Chloride concentrations in runoff, which usually approximate 20 mg/L, may reach 5 000 to 15 000 mg/L in snowmelt and spring runoff. Chlorine levels measured during periods of winter thaw in Black Creek in Metropolitan Toronto exceeded limits for domestic and industrial use and wildlife [9]. As indicated in Section 1.4.2, chloride from road salts represents a significant fraction of the total reaching Lake Ontario.

High concentrations of heavy metals have also been measured in surface runoff; for example, lead concentrations 210 times those in sanitary sewage have been recorded.

Snow falling on urban roads accumulates contaminants such as oxygen demanding substances, oils, salts, heavy metals, particulate matter such as clay and sand, litter, and often garbage [10], and direct disposal of polluted snow into receiving waters can result in water quality deterioration. Data collected by the Ministry of Environment shows that concentrations of all pollutants are higher in snow collected from more heavily travelled roadways.

Concern about the effect of deicing chemicals, and effects of disposal of polluted snow, has been reflected in a recent Ministry of Environment publication [10].

1.2.3 Pollution from illicit drainage into separated storm drainage systems

Concentrations and loadings for surface runoff in Table 2 are based on systems in which all dry weather flow reaches the sewage treatment plant via the sanitary sewerage system. This situation does not always exist, because through ignorance or deliberate misuse of the separated system:

1) sanitary building drains may be connected to the public storm sewer;
2) fixtures may be connected to the storm drainage system inside a building;
3) materials, e.g., crankcase oil, may be dumped into the nearest catch basin.

The results of these actions are discharge of untreated dry-weather flow from a storm system and/or increased pollutant loads when deposited materials are scoured from the storm system in wet weather.

Control of illicit discharges is a real difficulty in the management of separated systems. Toronto has found it almost impossible to properly police a separated system. An investigation of 600 houses in the city revealed 80 illicit connections [11]. A similar investigation in East York, involving 1000 houses in a separated sewer area 30-50 years old, found that about 5% had improper connections to the storm system. In about one-half of these cases, where sanitary building drains were connected to public storm sewers, corrections were made. Disappointingly few of the remainder, most of which consisted of basement fixtures connected by householders to the nearest drain, have been corrected [12].

1.2.4 Effects of overloaded sewerage systems

Many communities today face serious drainage problems resulting from the combined effects of upstream development, redevelopment, and inflow and infiltration.

Overloading of trunk sewers as a result of upstream development is an obvious direct effect of urbanization. A second indirect effect is land use change in areas close to the centre of cities. Areas that were originally occupied by low-density uses are now used for commercial and high-density residential use. In these areas, land surfaces become virtually 100 percent paved, and storm runoff is complete and immediate -- exceeding the capacity of drainage systems originally designed for low-density development. Many Ontario communities with old combined systems have flooding problems resulting from inadequate capacity compounded in some cases by structural inadequacy. Communities where such problems have been reported include Campbellford, Chatham, East York, London, Napanee, Ottawa, Sarnia, Thunder Bay, Toronto, and Wallaceburg.

Burlington and Windsor are examples of cities with overloaded storm drainage systems [3]. Street and basement flooding result in property damage, inconvenience, and health hazards when sewage is involved. In addition, overloading may cause structural damage to basement walls and floors in areas where check valves have been installed to prevent basement flooding. In these situations, high hydrostatic pressure, capable of causing damage, builds up when groundwater cannot escape because sewers are surcharged. This situation has occurred in the O'Connors Hills area of the Borough of North York [13] and in Scarborough, where, as a result of extreme summer storms in 1977, 191 houses in one area suffered flooding and 78 basements were structurally damaged [14].

Infiltration originates with leakage into old or poorly constructed public sewers or building connections. Improper connections of roof and foundation drains are major sources of inflow. The effects of inflow and infiltration include street and basement flooding from sanitary systems with attendant inconvenience, property damage and health hazards. Burlington, Kingsville, Leamington, Lincoln, Sault Ste. Marie, Smiths Falls, and Thorold are examples of communities where these problems are serious. The City of Kitchener has experienced problems because of footing drain connections to sanitary sewers [3]. In 1969 sewage flow at Cornwall was less than recorded water consumption; by 1974 the flow had increased to 167 percent of the recorded flow. This increase was interpreted as being due to rainfall and groundwater infiltration [15], with the result that expansion of the wastewater treatment plant became necessary. In Woodstock, measured infiltration in trunk sewers and at the treatment plant reached 9-17 m^3/ha•d (1000 - 1800 gallons per acre per day), and inflow values of 7.5 m^3/ha•d (800 gallons/acre•d) were recorded; these values may be compared with an average sewage flow of 10 m^3/ha•d (1100 gallons/acre•day) [16].

A number of "separated" systems with flooding problems are in fact only partly separated because of interconnections, diversions, and storm water connections. The sewerage system of St. Thomas was originally designed with separate sanitary systems, but in the early part of the century storm drainage problems resulted in connections from street and building drains, turning part of the system into a combined system [6].

Similarly, many interconnections exist in the Riverside area of the City of Windsor. In some parts of this area sanitary sewers are laid below storm sewers with common manholes separated by plates which, because of improper replacement, missing plates, etc., permit storm flow to enter the sanitary system [3].

Transmission and treatment facilities also experience the effects of flows for which they are not designed, in the form of overflows or by-passes from wastewater treatment plants and pumping stations. Examples include Kingston, where wet weather peak flows are five to seven times dry weather flow, and Sault Ste. Marie, where excessive peak flows are measured at the treatment facility [3]. The effects of excessive peak flows and overflows are particularly acute in communities where advanced treatment is required, such as Stratford.

The extent and magnitude of by-passes from municipal sewerage systems are indicated by the results of a recent study conducted in four Ontario communities [17]. The recorded by-pass activity occurred principally at treatment plants, and usually followed precipitation. In some instances by-passing resulted from poor design and operation or mechanical breakdown. Sewers in three of the four Cities -- Brantford, Dundas, and Waterloo -- are completely separated. Those in Aurora are about 10% combined. Figure 2 shows that sanitary sewage by-passes occurred 0.6 to 4.3% of the time, that 0.02 to 1.7% of total sewage flows were by-passed, and that these flows represented as low as 0.02% and as high as 6.1% of the pollutants discharged in the secondary effluent. The study period extended from the fall of 1973 to the summer of 1975.

The composition of by-passed material is indicated by Table 3. In general, by-passed flows resemble raw sewage.

1.3 Problems in Developing Urban Areas

In undeveloped areas, most rainfall soaks into the soil, from which it evaporates or penetrates downward to maintain or raise the groundwater table (see Figure 3). Stored groundwater drains slowly into lakes and streams, maintaining stream flow in dry periods. When ground surfaces are paved and surface litter is removed, less water enters the soil, groundwater levels drop, and dry weather stream flows decrease. In Aurora, groundwater levels and base flows in streams have been reduced as a

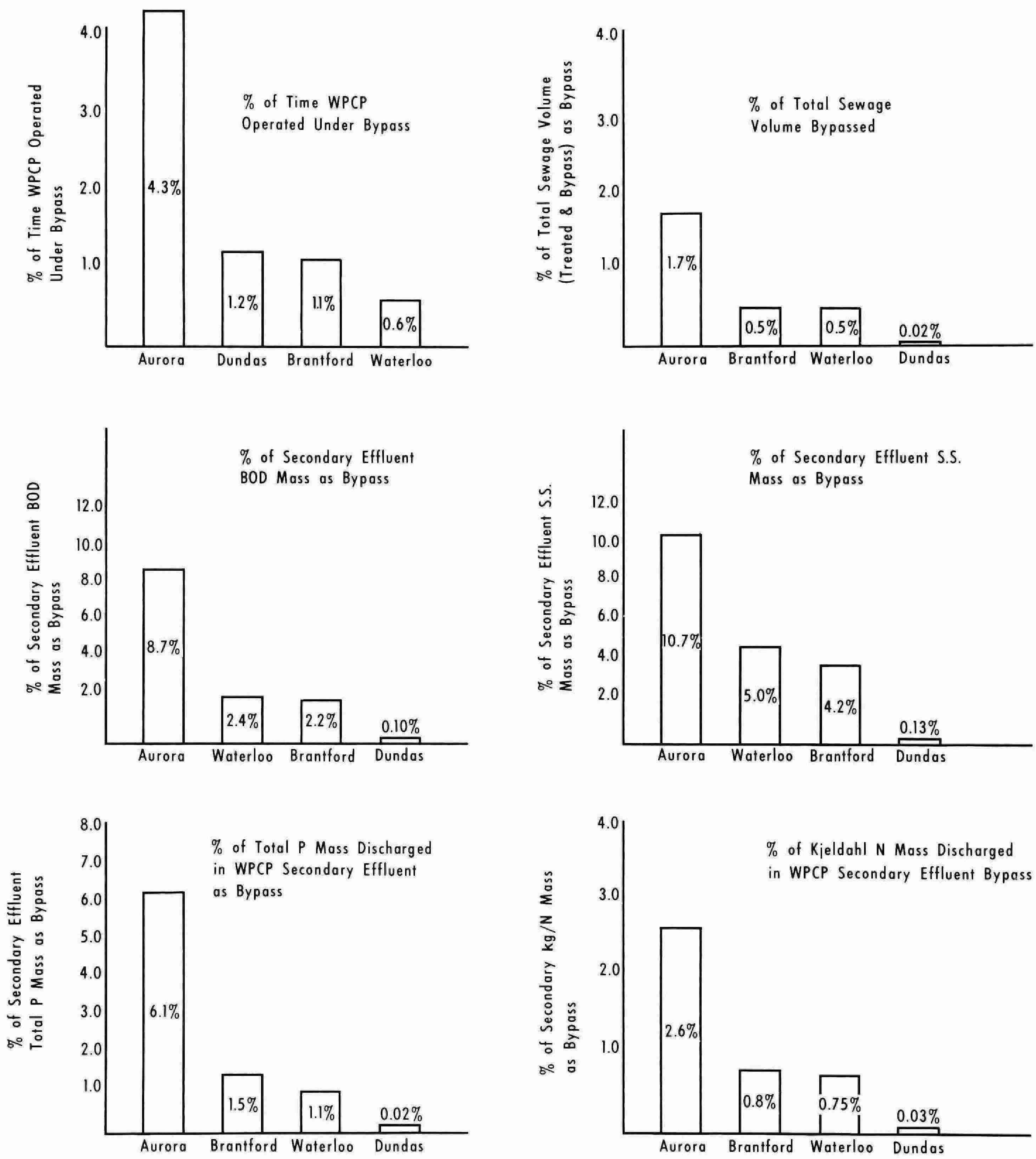


FIGURE 2. SANITARY SEWAGE BY-PASS MAGNITUDE [17]

TABLE 3. SANITARY BY-PASS QUALITY AT WPCP

	BOD (mg/L)			SS (mg/L)			Total P (mg/L)			Kjeldahl N (mg/L)		
Municipality	By-pass	Raw	Final	By-pass	Raw	Final	By-pass	Raw	Final	By-pass	Raw	Final
Aurora	142	114	30	412	256	63	5.0	4.6	1.5	38	36	24
Brantford	96	179	20	224	259	24	4.2	5.7	1.3	17	22	9.5
Waterloo	129	230	20	240	232	17	3.1	5.0	1.0	16	18	4.5
Dundas	108	69	27	239	87	45	5.8	4.7	3.8	30	22	19

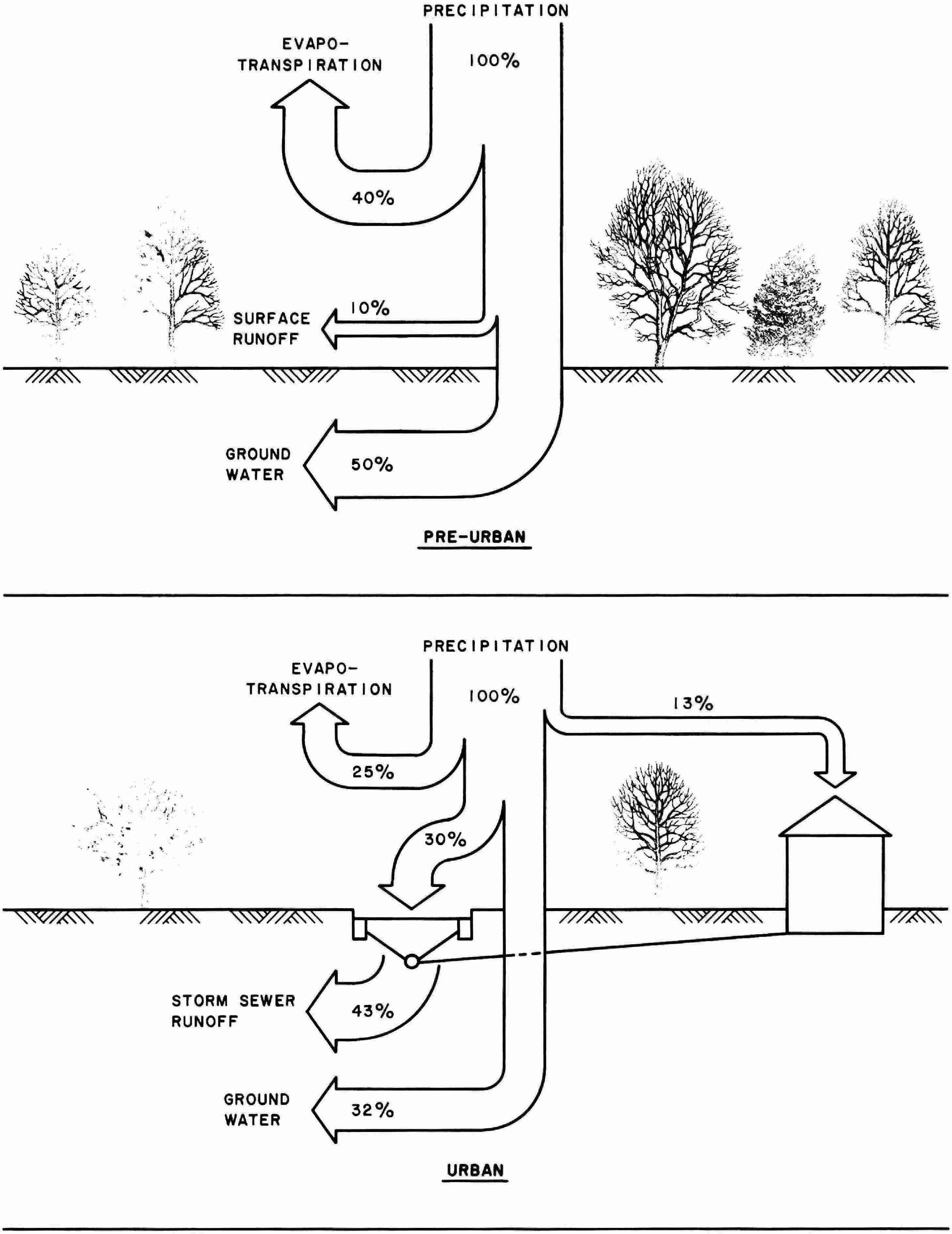


FIGURE 3. HYDROLOGIC CHANGES RESULTING FROM URBANIZATION

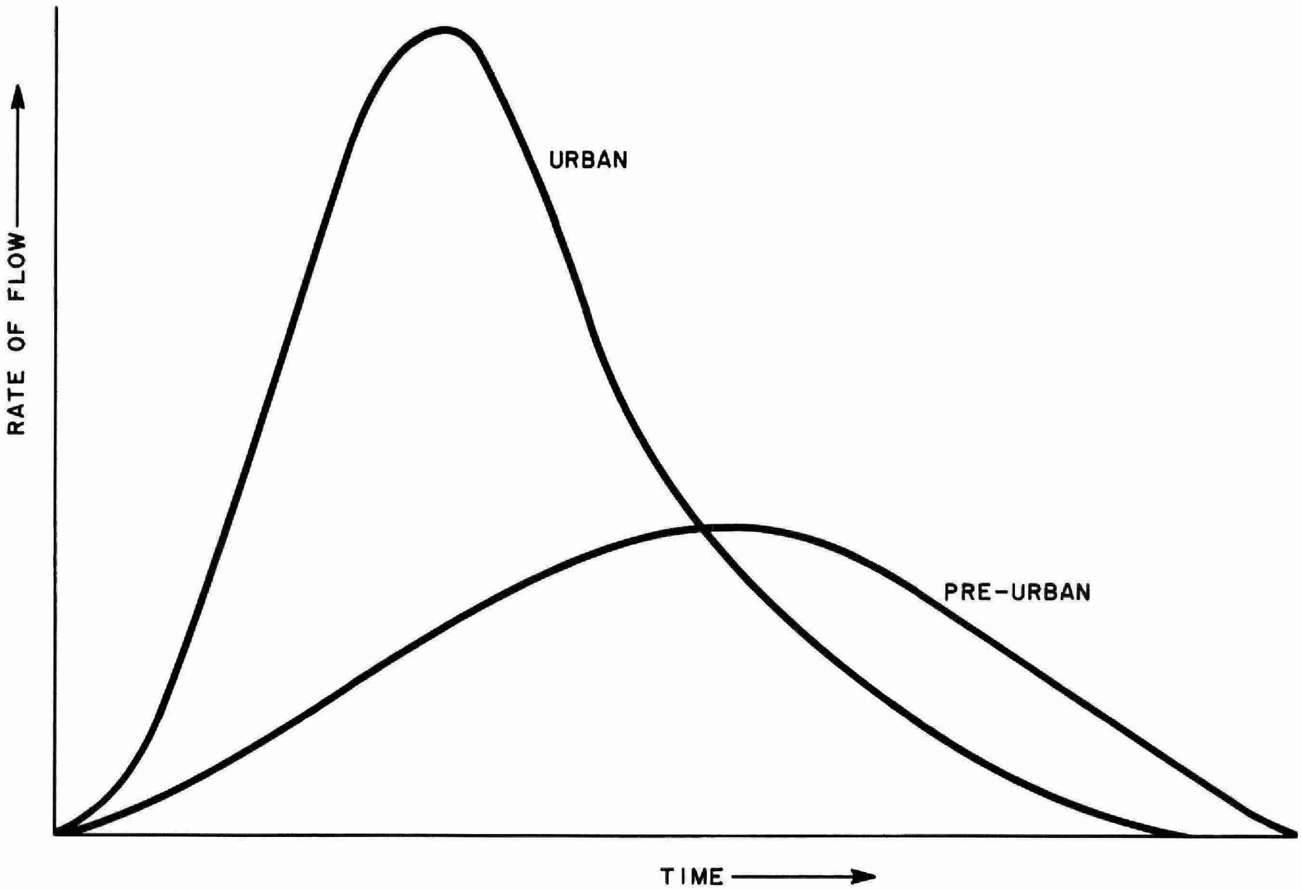
result of reduced groundwater recharge, construction of municipal services, and communal and domestic wells.

Surface runoff increases following urban development; flood volumes have been observed to increase by 1.5 - 2 times, and flood peaks two to five times [18]. These effects are illustrated by Figure 4. Storm drainage systems are installed to remove this excess water from urban surfaces. When these systems are discharged into local streams and the capacity of these streams is exceeded, the streams are straightened, deepened, widened or enclosed, and the problem is passed on downstream. Downstream solutions become more difficult and costly because structures such as bridges and culverts must be enlarged, and because flood plains are occupied by costly construction that must be protected.

Examples of the effects of upstream development include: a small drainage ditch in Timmins, which formerly carried approximately 0.6 m^3/s (20 cfs), must now accommodate 11 m^3/s (400 cfs); and, in the Metropolitan Toronto area, upstream development during the period between design and construction of channelization projects regularly results in the design capacity of the channels being exceeded in small storms. Highland Creek in Scarborough has been transformed over the past 25 years from a rural to a highly urbanized watershed. Damage caused by flooding as a result of two major storms in August of 1976 exceeded $1.3 million [19].

Not only are local flooding problems 'solved' at the expense of someone downstream, but often in the process, streams are altered for the worse and ponds, marshes, and other water bodies that might have been environmental assets are filled.

Another important problem in developing areas is soil erosion, which results when surface cover is stripped and the exposed soil is subjected to increased surface runoff rates. The effects include loss of valuable soil, turbidity in lakes and streams, and sediment accumulation in lakes, storage basins, and sewage treatment plants. The following values indicate how urbanization may change rates of soil loss (kg/ha•annum and lb/acre per year) from previously undeveloped land [20,21,22,23].



URBANIZATION INCREASES PEAK FLOWS AND RUNOFF VOLUMES
(THE AREA UNDER THE CURVES)

FIGURE 4. EFFECTS OF URBANIZATION ON VOLUME AND RATES OF SURFACE RUNOFF [18]

	kg/ha•annum	lb/acre per year
Forest land	35 - 670	30 - 600
Cultivated agricultural land	670 - 45 000	600 - 40 000
Exposed construction site	34 000 - 500 000	3 000 - 450 000
Developing urban area	560 - 13 500	500 - 12 000
Developed urban area	190 - 960	170 - 860

Downstream from developing areas, soil losses and resulting turbidity and sediment accumulation are caused by mechanical erosion due to increased stream flows. For example, many segments of water courses in Metropolitan Toronto exhibit considerable bank instability and require measures to control erosion. Several areas in Oshawa have experienced

serious bank erosion resulting in increased sediment loads in streams, and minor stream erosion problems have also been reported by the North Grey Conservation Authority.

Increased stream flow may result from increased runoff rates from a tributary basin, or from changes in timing of flood peaks from the basin.

1.4 Special Studies

This section examines studies of two areas -- the Thames River Basin and the Great Lakes -- which provide examples of a number of the problems discussed in previous sections.

1.4.1 Thames River Basin

A water management study of the Thames River Basin [24] identified and/or discussed the following urban water quality problems and their effects:

- suspended solids in storm water and combined sewage overflows;
- increased bacterial counts due to combined sewer overflows;
- the importance of unrecorded urban runoff, including sewage by-passed during storms, as a critical source of organic wastes from Woodstock, Chatham, Stratford and London;
- the persistence of troublesome levels of aquatic weed growth even after phosphorus removal to meet Great Lakes standards is achieved (due largely to augmentation of the remaining load by inputs from untreated urban runoff);
- 80% of the annual phosphorus contribution from Stratford originates from sewage by-passes, urban runoff, and minor point sources.

1.4.2 Great Lakes

The 1975 Report of the Great Lakes Water Quality Board of the International Joint Commission states: "Combined and storm sewer problems continue to be significant causes of water quality impairment in the 'problem areas' identified in this report". The report identifies two areas in Ontario, Toronto Harbour and Collingwood Harbour, where excessive bacterial counts and algae growth respectively are attributed to combined

sewer overflows. Other areas where such problems are identified include Hamilton Harbour and Thunder Bay.

Sixteen storm water outlets and 25 combined sewer overflows discharge into the Toronto Central Waterfront area [25]. High coliform counts in several areas are attributed to combined overflows, and sediment accumulations in the harbour are considered to be reservoirs of nutrients, heavy metals, and PCB's. The Don River contributes 98 percent of the sediment and about 75 percent of the BOD and phosphorus entering the harbour. The Toronto Harbour Commission has borne a significant increase in maintenance dredging costs as a result of the developmental activities in the Don River Basin.

Road salt, most of which is drained into storm sewers and stream channels by snowmelt or runoff, accounts for 20 to 40% of the total chloride input into Lake Ontario [26].

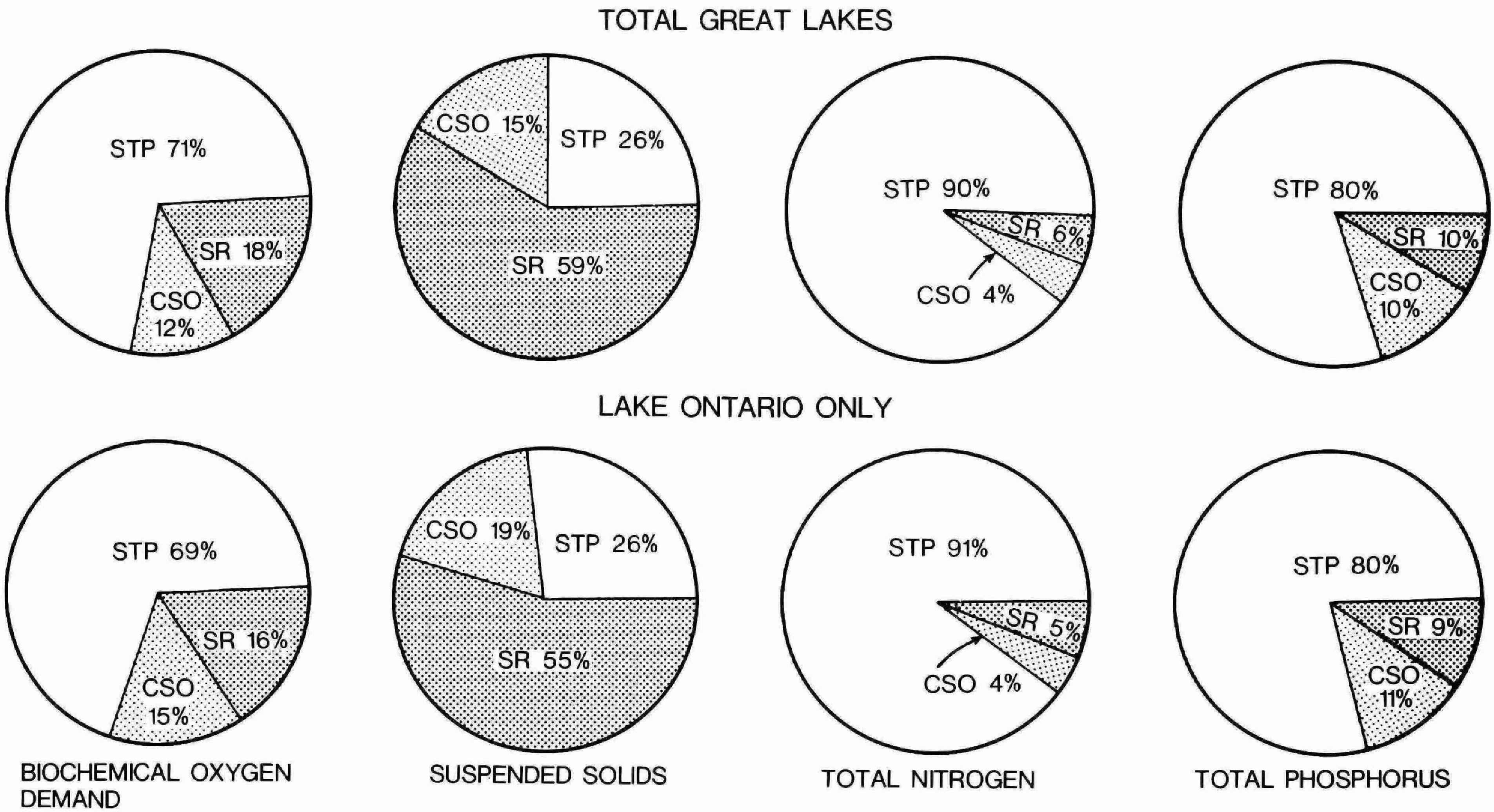
As well, urban wet-weather sources contribute significantly to accelerated eutrophication resulting from excessive phosphorus loadings. Municipal discharges account for approximately one-third of the total phosphorus discharged to the Great Lakes from Ontario [27].

Figure 5 [5] indicates relative contributions to the lakes from three municipal sources -- treatment plant effluents, combined sewer overflows, and surface runoff. The latter two sources will, after targets for phosphorus removal at municipal plants are met, account for more than 20% of annual loads from Ontario municipal sources, or over 7% of total loads from Ontario. When only Lake Ontario is considered, combined sewer overflows and surface runoff account for over 13% of total loads from Ontario.

The impact of combined sewage and surface runoff is greater if only wet weather loads are considered. Figure 6 [5] indicates that in wet weather these sources discharge BOD and phosphorus loads that are more than twice, and suspended solids loads that are nearly five times wet weather sewage treatment plant discharges.

1.5 Costs of Urban Drainage Improvements

No detailed overall estimate has been made of costs for remedying urban drainage problems; however, the following indicates the order of magnitude of expected costs:



LEGEND - STP - sewage treatment plant effluent
CSO - combined sewage overflow
SR - surface runoff from separately sewered areas

FIGURE 5. RELATIVE CONTRIBUTIONS TO THE GREAT LAKES FROM MUNICIPAL SOURCES IN ONTARIO [5]

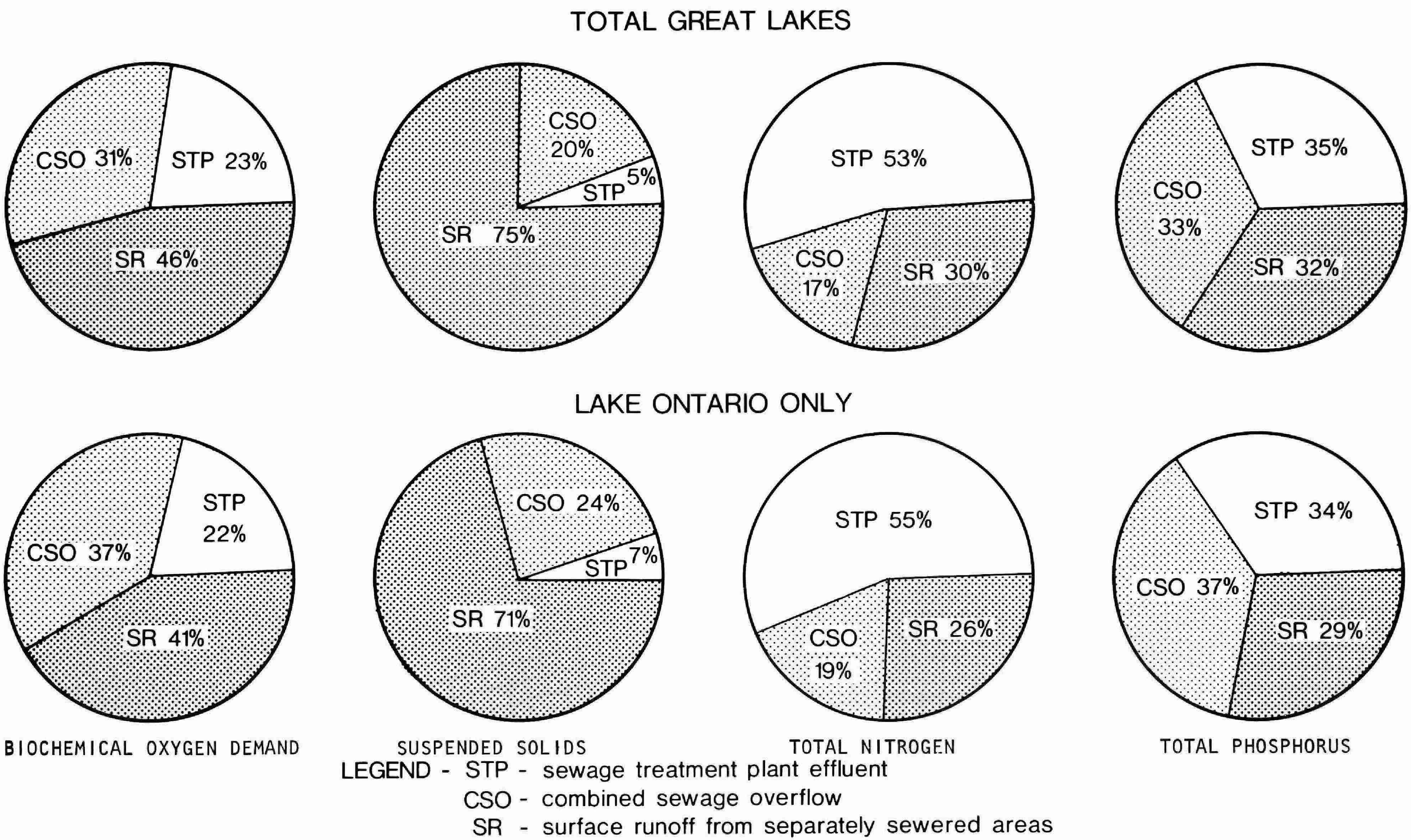


FIGURE 6. RELATIVE CONTRIBUTIONS OF WET WEATHER LOADS TO THE GREAT LAKES FROM MUNICIPAL SOURCES IN ONTARIO [5]

- Approximately 38% of the population of Ontario is served by combined sewers. Many of these sewers are inadequate because of condition or capacity. An indication of replacement cost is an estimated cost of $45 120 per ha ($18 260 per acre) [ENR 2000] to separate combined sewerage systems in the United States [28]. Applied at ENR 2500 to the 28 730 ha (71 000 acres) served by combined sewers in Ontario [3] this corresponds to a total cost of $1.3 billion.
- According to a recent estimate the cost of providing storage and treatment could vary from $0.23 billion for 50 percent control to $0.55 billion for 75 percent control of wet weather pollution in the Great Lakes Basin [3].

These estimates exclude detailed consideration of factors such as required trunk sewer construction, the need for replacement of overloaded storm drainage systems, and the possibility that in some communities alternative approaches may cost less than combined sewer separation. Nevertheless, they suggest that the overall cost of solving flooding and water pollution problems associated with existing urban drainage systems in the Great Lakes Basin will approach or exceed $1 billion.

1.6 Institutional Problems

Earlier sections of this chapter have dealt with the nature, magnitude and costs for correction of technological problems related to urban drainage. Another class of problems arises from the conflicting policies and objectives of various agencies with interests and responsibilities related to urban drainage.

All agencies of government have the common goals of preserving and improving the quality of the environment for present and future uses, maintaining and upgrading the standard of living, as well as protecting life and property from damages. In terms of water resources management, these goals can be translated into the following specific objectives, which can be violated by inadequate or improperly controlled urban drainage:

- prevent or correct hazards to health,
- maintain surface water quality for recreation, aquatic life, and other uses,
- maintain adequate base flow in streams for all water uses,
- enhance groundwater quality and quantity,
- improve water quality where adversely affected by pollutant discharges,
- reduce erosion and sedimentation in streams for the preservation of aquatic biota habitat, bank protection, and preservation of stream capacity,
- provide convenient access to property during and following rainfall or snowmelt events,
- prevent downstream flood damage,
- prevent basement flooding,
- interface land use planning with water resources management,
- achieve the above objectives at reasonable costs.

When the objectives of individual agencies are pursued in isolation, solutions that meet their individual aims, if not considered in a broader context, can create or aggravate other problems related to urban drainage. Examples are: piece-meal land development within a watershed by individual municipalities and developers, resulting in downstream flooding, construction in flood plains, and ecological effects of stream channelization; creation of inflow problems by connection of foundation drainage to sanitary sewers in order to prevent or solve basement damage problems; and poor coordination of infiltration prevention programs in communities where public sewers are carefully constructed and inspected but construction of building sewers, which are governed by building codes, is not well controlled.

Although these problems are recognized, this manual does not attempt to propose solutions. Hopefully solutions to institutional problems will evolve as urban drainage policies in Ontario are developed.

1.7 Water Quality Impacts

Previous sections have described the quality of waters discharged from urban drainage systems, with only limited discussion of the impacts that pollutants from these sources may have on the receiving water,

assuming that many readers will be familiar with these effects. This section is included for those readers who may wish further information on the potential impacts of urban pollutants upon receiving waters.

1.7.1 Nutrient effects

As in terrestrial plant culture, nutrients act as "fertilizers" and promote the excessive growth of aquatic plants and algae. In the presence of abundant sunlight and with suitable physical conditions of temperature, substrate, and current, growth of aquatic plants is limited only by space and nutrient levels in streams, ponds or lakes. Nutrient inputs can result in unsightly accumulations of aquatic plants and algae causing oxygen concentrations to fall below critical levels.

1.7.2 Oxygen demand effects

Oxygen demand is commonly measured by the BOD test, which indicates the potential effect of carbon-containing compounds. Additional demands for oxygen may be imposed by unoxidized nitrogen forms (organic and ammonia nitrogen). The decomposition of organic material is achieved through the action of bacteria. The respiratory demand exerted by large bacterial populations can reduce oxygen levels in the receiving stream to concentrations critical to fish and aquatic life. Bacterial populations decrease with progress downstream as their food supply is gradually used up, so that oxygen depletion due to bacterial respiration is usually a localized phenomenon.

Superimposed on the oxygen demand associated with organic decomposition is the photosynthesis-respiration phenomenon. While oxygen may be added to the stream by aquatic plants during daylight hours, respiration by these same plants during darkness often results in temporary oxygen depletion.

Where oxygen is depleted to critical levels in surface waters, fish are asphyxiated and desirable fish-food organisms are eliminated. Under less critical conditions, reduced dissolved oxygen levels represent a stress on fish. The presence of other stress factors such as toxic materials or high temperature can combine to be lethal, or to severely reduce fish reproduction.

Cause-and-effect judgements of water quality problems frequently involve complex inter-relationships. For example, Figure 7 [24] shows the

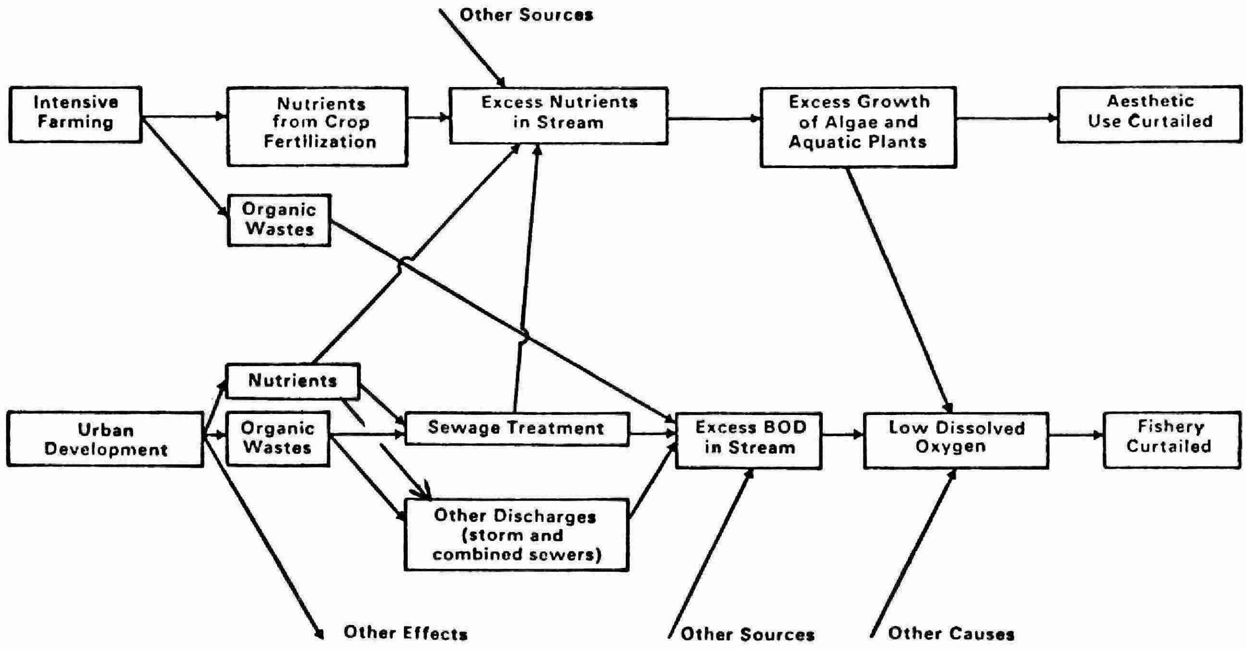


FIGURE 7. EFFECT OF URBAN AND AGRICULTURAL ACTIVITIES ON AESTHETICS AND FISHERIES IN THE THAMES RIVER BASIN [24]

eventual effect of urban and agricultural activity on aesthetics and fisheries in the Thames River Basin.

> "In the example, nutrients from urban and rural sources overfertilize the stream, resulting in profuse growths of algae and aquatic plants in summer when other conditions are favourable for growth. Excess aquatic plants are unsightly and adversely affect the aesthetics of the water course.
>
> "Water quality problems are most severe in summer months when a combination of natural and man-made factors magnify adverse water quality effects. Lack of river bank vegetative cover, increases in sunlight enèrgy and excess quantitites of plant nutrients provide ideal conditions for aquatic cover and algae growth. The steady build-up of plant biomass is typically accompanied by a steady recession of natural streamflows which magnifies the effects of the biomass, and also provides less water to dilute waste materials continuously discharged from urban and rural areas.
>
> "Water temperature increases in summer and causes increased respiration rates of all organisms in the water. At the same time, oxygen saturation values decrease with increasing temperature, so that less oxygen is available in the water for the competing organisms.
>
> "The combined meteorological, physical, and biological conditions described above can lead to crucial stress conditions for fish life. Symptomatic of extreme stress conditions in the aquatic environment is the sudden localized appearance of large numbers of dead fish". [24]

1.7.3 Sediment effects

Sediments include suspended solids in combined sewage, surface runoff, and soil materials eroded from construction sites and stream channels. Suspended solids in a watercourse render the water aesthetically displeasing because of the resultant muddy appearance, and block stream channels, fill reservoirs and can endanger fish. Subsequent deposition of the material can foul the stream bottom and fish spawning beds and smother bottom organisms that support stream productivity. Organic solids from sewage treatment plants, storm sewers, and combined sewers, often settle out to form sludge banks in slow moving sections of the river, where they can represent a reservoir of oxygen demand, bacteria, metals, nutrients, and organic chemicals which can affect overlying waters or be swept downstream in storm periods.

1.7.4 Effects of salt

Excessive salt concentrations, generally associated with ice control on streets, can impair the value of ground and surface waters for some industrial uses, affect the palatability of groundwater, and cause or increase stratification of lakes thereby contributing to accelerated eutrophication. The maximum concentration of sodium (20 mg/L) for persons on low sodium diets is being exceeded in an increasing number of communities in the eastern U.S. [29].

1.7.5 Bacteriological effects

High bacterial counts are of particular concern in evaluating the safety of bathing waters. Coliform and fecal coliform bacteria are used to evaluate water quality in relation to public health. Although coliforms are not normally regarded as pathogenic, their presence in water indicates the potential presence of scarcer and much more difficult to isolate pathogenic organisms such as those causing typhoid fever, dysentry and cholera. Other pathogenic bacteria detrimental to public health can cause a variety of physiological disorders, such as minor skin infections, or ear, eye, nose, and throat infections. As Table 2 indicates, contamination by combined sewage overflows or surface runoff can result in beach closures because bacterial concentrations in these wastewaters exceed bathing water standards.

1.7.6 Other effects

Heavy metals and persistant organic substances in combined overflows and surface runoff are potential threats to aquatic organisms and to higher organisms, through accumulation in the food chain. Although metals remain largely insoluble and accumulate in sediments, under soft water conditions, for example, copper, lead, zinc, and cadmium could be dissolved and become toxic to some aquatic organisms.

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2 URBAN DRAINAGE PLANNING

This chapter outlines a planning framework for urban storm water management problem solving. This general planning approach considers the various goals and objectives of different sectors of society affected by urban development and urban-related drainage problems, enables the identification of suitable alternatives and an evaluation of their effectiveness, and results in an acceptable and implementable plan of action. This approach is applied to two important urban drainage considerations: minimizing problems resulting from new land development, and alleviating problems in existing municipalities.

2.1 The Purpose of Planning

Unless comprehensive planning strategies are introduced, continued urbanization in the already urbanized areas of southern Ontario will aggravate existing flooding and pollution problems in local watercourses and in the Great Lakes. The greater population and the trend towards more leisure time will increase demands for water-based recreation and control of pollution problems.

The past lack of coordination between storm water management practices and overall land use planning has led to the costly and complex pollution and drainage control correction programs that many municipalities are now undertaking. Expenditures on these preventative and remedial flood and pollution control measures must be justified in a time when there is general restraint on spending in the public sector of the economy. These projects must compete for funds with other projects which also have some social benefits and contribute to the health, safety, convenience, and welfare of the public. Integration of land use planning and urban drainage management could avoid many problems in the future.

Urban drainage is often treated in a piecemeal fashion. In new areas, drainage is considered a servicing problem with a limited objective of rapidly disposing of all runoff. In existing areas, studies to control specific problems such as basement flooding from inadequate drainage systems, often do not consider the broader scope of the problems or available solutions.

However, there is a trend in Ontario away from piecemeal considerations towards more comprehensive planning incorporating multiple

objectives, wider scope, and space and time implications. River basin planning in the Thames River watershed [1] and more comprehensive site planning in new urban areas are examples of this trend.

In Ontario, planning is often considered in the context of "The Planning Act", which governs municipal planning [2]. However, the more general definition is implied in this chapter to include the decision-making process at any level of government. The overall goal of planning implied in the Act, in the definition of an official plan and in the considerations for subdivisions, is to provide for the health, safety, convenience, and welfare of present and future inhabitants.

2.2 General Planning Considerations

This section will discuss general concepts of environmental, flood control, and economic planning, as well as outline some of the characteristics of drainage systems.

2.2.1 Urban drainage concepts

Following are general concepts that apply to the planning and design of urban drainage systems.

a) Urban runoff systems compete for space and resources with other land and water uses. In order to provide for protection of the designated function of a parcel of land, and to minimize downstream flood and environmental damages, design of urban runoff systems should be carried out at the earliest practical stage of site development. Site drainage designs should be based on, and supportive of, a master drainage plan for the entire watershed. Master plans for storm drainage should be developed and maintained in current form at all times for each drainage basin. The planning of drainage facilities should be coordinated with planning for open space, for sanitary sewage disposal, and for transportation. By coordinating these efforts, new opportunities may be identified which can assist in the solution of drainage problems.

b) The natural planning boundary for water resource systems is the watershed. The watershed acts as a single integrated system during runoff events and is consequently a logical planning unit for flood control. Conservation authorities in Ontario are organized on a watershed basis for flood control planning and the

operation of flood control systems. Environmental planning as well is carried out successfully on a watershed basis since watercourses act as a natural disposal system for the conveyance and assimilation of sewage and runoff from all land uses in the watershed.

c) The pipe system or convenience system is only a sub-system of the total drainage network in the urban area. The system of roadway gutters, enclosed conduits, and connections to roof leaders is often called the minor or convenience system. At certain times the capacity of the minor pipe system will be exceeded. The surface drainage route or network which operates when the minor system is at capacity is often called the major drainage system.

d) Urban development and the associated drainage systems can drastically alter the natural hydrologic cycle both locally and downstream of the watershed. Consideration should be given to maintaining the natural predevelopment hydrologic characteristics where possible and where beneficial. These characteristics include groundwater recharge, and the maintenance of water tables, peak runoff volumes and rates, and base flow in dry periods. High density development may preclude achievement of some aspects of this goal, since opportunities for mitigating works may be reduced; however, where attainable at reasonable costs, natural drainage principles should be adopted.

e) Consideration should be given to the possible damage due to surcharging of the pipe system from connection of foundation tile drains for residential housing.

f) Storm water runoff can be stored in detention and retention reservoirs. Such storage reduces the drainage capacity required, thereby reducing downstream expenditures. Acquisition of parkland adjacent to drainage ways will provide areas where storm runoff can spread out and be stored for slower delivery downstream.

g) Natural drainage channels and associated ravines and floodplains in urban areas have a high recreation use potential - if they are

maintained in a relatively natural state and protected from degradation caused by increased rates and volume of runoff erosion, and flooding.

2.2.2 Flood control objectives and criteria

One of the primary functions of urban drainage design is to protect against flooding damage. A recent report [3] presents design guidelines which recognize the objective of urban storm drainage to be "the elimination or minimizing of flood damage and hazard under long-term storm conditions, and the removal of street surface flows under short-term conditions to the extent required to provide a reasonable level and frequency of convenience and safety for pedestrian and traffic use."

Control of urban runoff may be only part of a watershed flood control management plan in view of the small percentage of urban land draining to some watersheds and the fact that many existing urban areas are in flood plains subject to frequent flooding.

The criterion for flood protection in the Province of Ontario is the peak flow generated by a regional storm or by the 100-year storm, whichever is greater. This is intended as a practical expression of the degree of flood protection that should be provided to the citizens of Ontario. A regional storm is defined in the Ontario regulations developed by the various conservation authorities, under the aegis of the Conservation Authorities Act [4].

Urban drainage systems should be designed so that no damage to property results during the design flood. The total flow may be carried by a combination of enclosed conduits, open channels and overland flow, provided that the objective of flood protection is fulfilled.

For drainage areas greater than 2.6 km^2 (1 mi^2), the flood plains of the watercourses should be set aside, outside the proposed building lots, for carrying the regional flood flows. For drainage areas between 1.3 - 2.6 km^2 (0.5 1.0 mi^2), building lots may encroach on the flood plain, provided that the structures are built outside the flood plain limits. Flood plains are generally not outlined for drainage areas smaller than 1.3 km^2 (0.5 mi^2) since it is considered that traditional urban drainage concepts apply, as determined by each individual municipality. However,

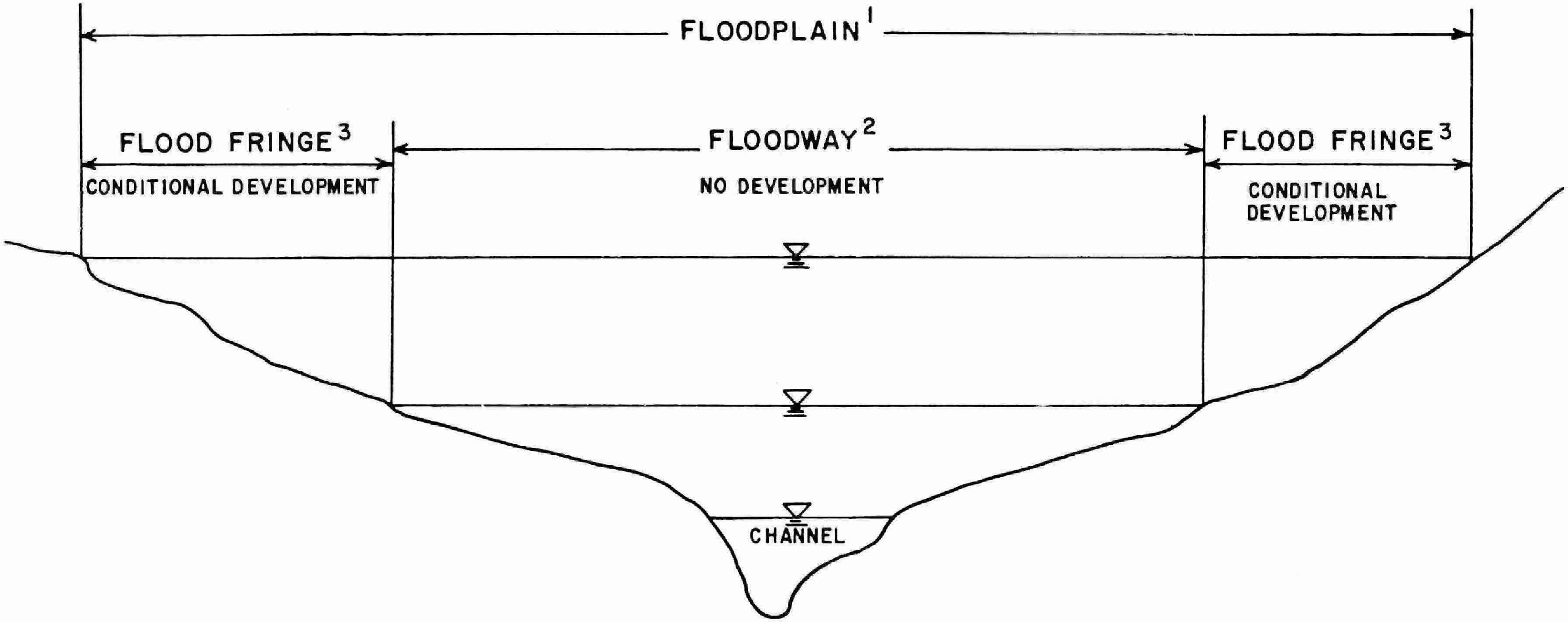
drainage design for these areas should still provide the same degree of protection to property and life as outlined previously, i.e., regional storm or 100-year storm.

The flood plain criteria are presently being reviewed by the Province of Ontario [27]. A "two-zone floodway-flood fringe" concept has been proposed for some areas (see Figure 8). Under the two-zone concept, development would be permitted in the fringe of the flood plain if special flood protection measures were adopted. Development would not be permitted in the floodway, i.e., the danger zone of the flood plain where deep and fast flowing waters would cause loss of life and severe damage to property. Present criteria used to define the flood plain in various areas of the province would remain in effect.

Conservation authorities in Ontario have been carrying out the mapping and the delineation of flood plains within the areas under their jurisdiction. The authorities make regulations which give them the powers of prohibiting or regulating development in areas susceptible to flooding. It is proposed that conservation authorities delegate jurisdiction for urban storm water management, for larger watersheds than in the past, to local municipalities, once the municipalities adopt storm water management plans that comply with the objectives of flood plain management and master drainage planning.

The objective of flood protection also applies to the design of bridges and culverts. Design to regional flood criteria is to be considered if, under regional flood conditions, a facility designed to normal criteria would increase flood damage or would have an unacceptable effect on upstream lands. The frequency of overtopping of the roadway should be commensurate with the acceptable level of inconvenience to the users of the facility and also the increase in maintenance costs.

In new urban areas, which are designed according to the above criteria for protection of property, the design criteria for the pipe or convenience system should reflect the degree of convenience required for the use of the area drained. For example, major highways, arterial roads, and local streets respectively would require decreasing levels of protection against minor flooding. In addition, foundations should be protected from surcharge conditions which might cause foundation failure or basement flooding, either by not allowing foundation tile drains to be



1. The floodplain would be defined by the Hazel flood, the Timmins flood or the 1 in 100 year flood, depending upon the location in the Province.
2. Floodway is defined as the danger zone in which no building or filling to be permitted.
3. Flood fringe is the area, where filling and development may be permitted if special flood protection measures are adopted.

FIGURE 8. THE TWO-ZONE FLOODWAY - FLOOD FRINGE CONCEPT

connected by gravity to storm sewers, or by ensuring that the pipe system does not surcharge under major storm conditions. A discussion of design criteria for the convenience system as well as methods for quantifying the relationship in design is provided in Chapter 3.

2.2.3 Environmental goals

2.2.3.1 Goals and objectives for surface water quality in Ontario. Ontario outlined its approach to water management in a recent publication "Water Management - Goals, Policies, Objectives and Implementation Procedures for the Ministry of Environment" [32]:

> "The surface waters of Ontario are put to many uses and each use has specific water quality requirements. Water quality must be managed, preserved, and restored where necessary to permit the greatest number of uses, based on the best interest of the people of Ontario. In addition, Ontario borders on inter-provincial and international waters, and the implications of the Province's activities must be considered in that context. For example, the Province has agreed that the revised Specific Water Quality Objectives contained in the Great Lakes Water Quality Agreement shall be used in environmental programs to achieve and maintain Great Lakes water quality. Moreover, under the Canada-Ontario Accord, Ontario will establish and enforce effluent requirements for specific industrial groups and pollutants, to be developed by the Federal Government in consultation with the provinces."

The province's main concerns are the protection of public health and the protection of fish and aquatic life. Accordingly, a single goal was established as the target for all surface waters in the province [33]:

> "TO ENSURE THAT THE SURFACE WATERS OF THE PROVINCE ARE OF A QUALITY WHICH IS SATISFACTORY FOR AQUATIC LIFE AND RECREATION.
>
> "The Provincial Water Quality Objectives represent the level of quality for many parameters which will ensure that the water quality is suitable for aquatic life and recreation. In general, water meeting this quality will be suitable for most other beneficial uses, including raw water supply for human consumption and agricultural use..... For the few exceptions in those locations where better water quality is required to protect other beneficial uses, the appropriate water quality objectives will be applied."
>
> "The Provincial Water Quality Objectives represent the revisions to the existing criteria and are based mainly on a review of the recommendations of the International Joint Commission and of the U.S. Environmental Protection Agency. The rationale for each Objective is contained in a separate report by the Ministry of the Environment."

Following the goal statement are five statements of policy on surface water quality management [33]:

"1. <u>Areas with Water Quality Better than the Objectives</u>

IN AREAS WHICH HAVE QUALITY BETTER THAN THE PROVINCIAL WATER QUALITY OBJECTIVES, WATER QUALITY SHALL BE MAINTAINED AT OR ABOVE THE OBJECTIVES.

"2. <u>Areas with Water Quality not Meeting the Objectives</u>

WATER QUALITY WHICH PRESENTLY DOES NOT MEET THE PROVINCIAL WATER QUALITY OBJECTIVES SHALL NOT BE DEGRADED FURTHER AND ALL PRACTICAL MEASURES SHALL BE TAKEN TO UPGRADE THE WATER QUALITY TO THE OBJECTIVES.

"3. <u>Effluent Requirements</u>

EFFLUENT REQUIREMENTS WILL BE ESTABLISHED ON A CASE-BY-CASE BASIS. IN ESTABLISHING EFFLUENT REQUIREMENTS, THE CHARACTERISTICS OF THE RECEIVING WATER BODY WILL BE CONSIDERED, AS WILL FEDERAL AND PROVINCIAL EFFLUENT REGULATIONS AND GUIDELINES WHERE APPLICABLE. THE EFFLUENT REQUIREMENTS SO DERIVED WILL BE INCORPORATED INTO CERTIFICATES OF APPROVAL (UNDER SECTION 42, ONTARIO WATER RESOURCES ACT) AND WILL SPECIFY BOTH WASTE LOADINGS AND CONCENTRATIONS.

"In implementing Pollicy 3 [sic], the acceptable loadings or concentrations will be based on the most stringent of receiving water assessments and federal or provincial effluent regulations or guidelines.

"Many considerations must be taken into account when establishing effluent requirements. The more important ones dealt with in the implementation procedures include discussions of non-point sources, the types of receiving water assessment techniques to be used, toxicity testing, and how the Federal effluent regulations and guidelines are to be applied. A detailed procedures on how to deal with the taking and discharge of cooling water is also provided [sic].

"Policy 3 represents an essential aspect of the proposed surface-water quality management approach, as it provides the basis for the systematic establishment of effluent requirements. The lack of such an approach was identified as a problem with the original guidelines. Moreover, the decision not to recommend receiving water or province-wide effluent standards was predicated in part on the basis that effluent requirements would be established as a legal requirement on a case-by-case basis.

"4. Hazardous Substances

Policy 4 requires the elimination of a short list of substances from all controllable sources and requires that care be taken in the release of a second list of substances of unknown but potential hazard.

"5. Mixing Zones

A MIXING ZONE IS DEFINED AS AN AREA OF WATER CONTINGUOUS TO A POINT SOURCE WHERE THE WATER QUALITY DOES NOT COMPLY WITH THE PROVINCIAL WATER QUALITY OBJECTIVES. TERMS AND CONDITIONS RELATED TO A MIXING ZONE WILL BE DESIGNATED ON A CASE-BY-CASE BASIS AND MAY BE SPECIFIED IN CERTIFICATES OF APPROVAL, CONTROL ORDERS, REQUIREMENT AND DIRECTIONS, OR APPROVALS TO PROCEED UNDER THE ENVIRONMENTAL ASSESSMENT ACT. THE SIZE OF THE MIXING ZONE SHALL BE MINIMIZED TO THE GREATEST POSSIBLE DEGREE AND UNDER NO CIRCUMSTANCES IS THE MIXING ZONE TO BE USED AS AN ALTERNATIVE TO TREATMENT."

2.2.3.2 Agreement on Great Lakes water quality. In 1972, following recommendations of the International Joint Commission, the government of Canada and the government of the United States signed the Agreement on Great Lakes Water Quality. The primary emphasis of the agreement was to rectify existing pollution problems in the lower Great Lakes. The agreement also called for prevention of further pollution due to population growth, resource development, and increasing use of water. The Great Lakes Water Quality Board in its 1974 report [6] drew attention to this provision and emphasized the need for effective water quality-related land use planning to meet the needs of future growth and development, consistent with the achievement of the water quality objectives. The agreement was renegotiated and renewed in 1978, strengthening the requirements for improvement.

General and specific water quality objectives for the lakes, interconnecting rivers, and nearshore waters were included in the agreement. The specific objectives were usually defined to ensure protection of the most sensitive use. In some cases, however, protection of the aquatic food chain, protection of the public water supply, or aesthetics were determining factors in setting acceptable levels for water quality parameters.

2.2.3.3 Assessment techniques. Appropriate assessment techniques for a given watercourse should include consideration of the Provincial water quality objectives and also the special characteristics of diffuse source pollution. These considerations include:

i) The specific pollutant parameters in runoff or combined sewer overflows most likely to affect the particular beneficial uses of the watercourse. These could include oxygen-demanding materials, algal and plant nutrients, suspended solids, dissolved solids, pathogenic organisms, toxic substances, persistent trace contaminants, oil and grease.

ii) The duration of pollutant runoff events and the time required to mix, transport, disperse, settle, and assimilate the pollutant in the environment. Some effects are of short duration, such as immediate increases in a soluble pollutant concentration in a river which is rapidly diluted by natural flow. Other effects linger long after the runoff event, such as the settlement of solid organic pollutants to form sludge banks which then exert an oxygen demand over time and slowly release nutrients to the overlying waters. The effect of time on storm runoff quality problems is shown in Figure 9.

iii) The pollutant washoff phenomena. It is generally recognized that a large storm runoff event will contribute a greater pollutant load to a receiving water than a small volume runoff event (all other factors being equal). However, runoff from the latter storm may contain a higher average pollutant concentration. The choice of a critical storm event should take into consideration the ability of the receiving water to assimilate the different loads and concentrations resulting from each type of event.

iv) The size and nature of the receiving water body, whether it is a small recreational lake, the Great Lakes, a small stream or a large river. Lakes tend to settle and store pollutant materials in sediments, in solution, and in the aquatic biota. The degree of recycling of the pollutant material in the lake can affect the importance of control measures. The effect of areal scale on storm runoff quality problems is shown in Figure 10.

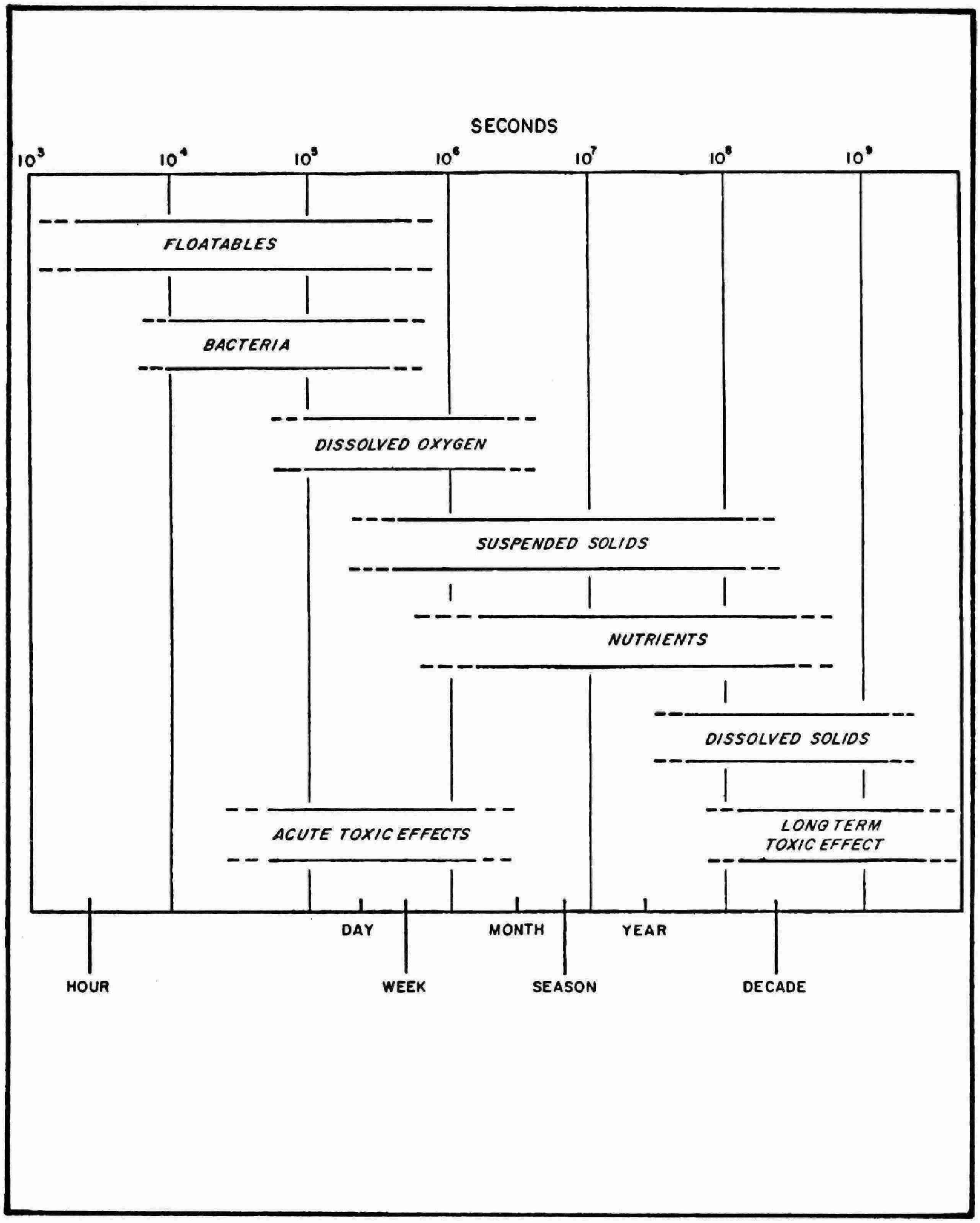


FIGURE 9. TIME SCALES - STORM RUNOFF WATER QUALITY PROBLEMS [28]

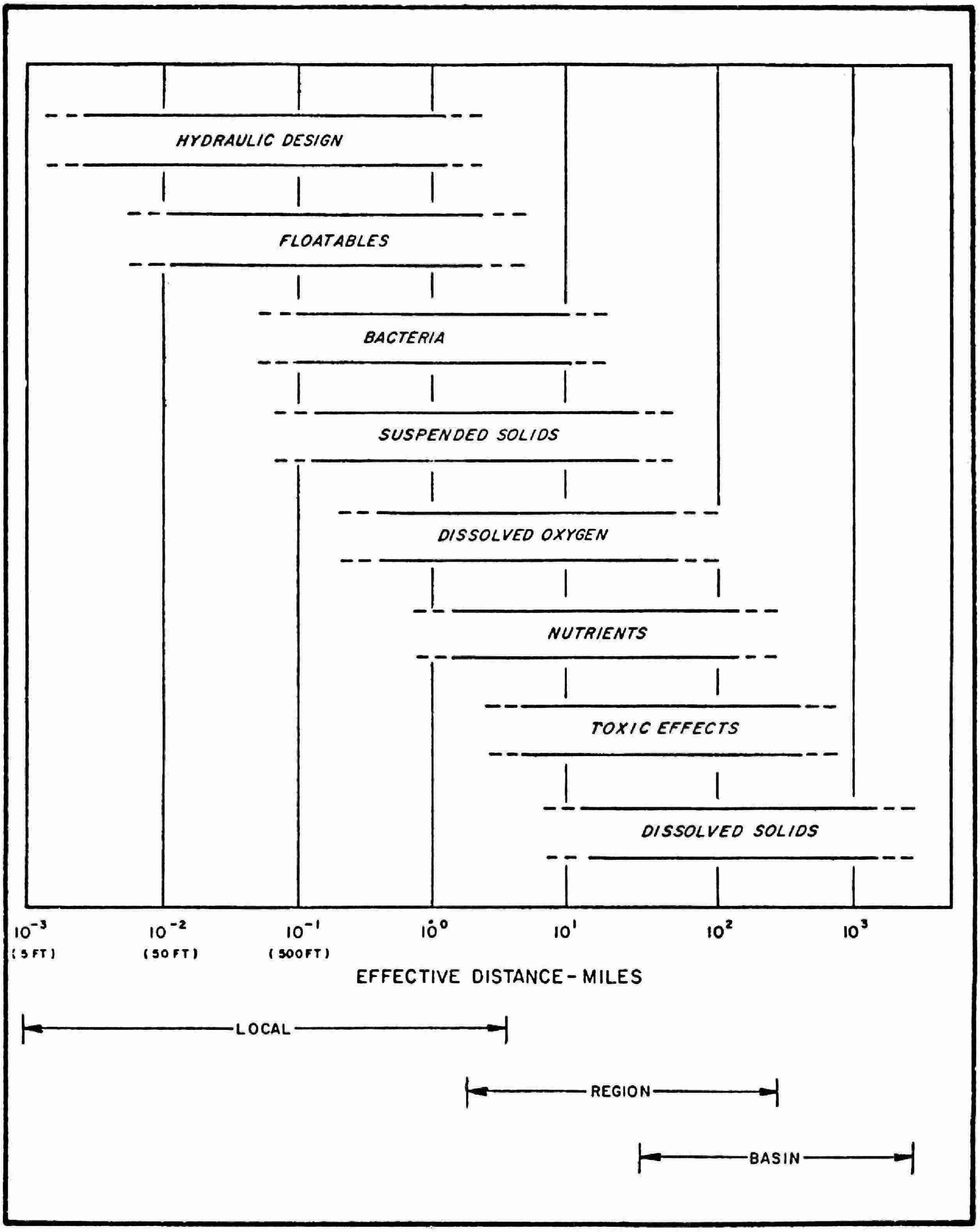


FIGURE 10. SPACE SCALES - STORM RUNOFF WATER QUALITY PROBLEMS [28]

v) The ability of a particular use to endure, survive, or be interrupted by runoff pollution events. Swimming in a river could be curtailed immediately following a runoff event until bacteria were transported, diluted or had died off. However, fish may not survive a short-term stress period caused by toxic substances or reduced dissolved oxygen concentration after a runoff event.

Ideally, evaluation criteria for water quality control should be based on receiving water response. Alternative statements of criteria could be in terms of effluent guidelines or specific control measures. These forms of criteria and their implications are discussed below.

2.2.3.4 Receiving water criteria. The use of criteria formulated in terms of environmental response is the ideal case since the criteria relate directly to the pollutant effect. A clear advantage of this approach is that the water quality criteria used to establish the existence of a problem are the same as the evaluation criteria used to compare alternatives. However, data needs can be extensive and modelling of the environmental response may be complex.

The form of criteria relating to performance over the long term or in response to a specified critical event is dependent on the factors discussed previously, and in turn governs the type of modelling required.

Statistical or probablistic criteria would be based on the number of occurrences and/or the total duration of violations of criteria for dissolved oxygen or pollutant concentrations in a given time period (month, year or season). Dissolved oxygen criteria formulated for the Thames River study [1] are an example of receiving water criteria which can be applied to continuous simulation cases.

Criteria for critical events would be based on maintaining a given level of quality (dissolved oxygen or pollutant concentration) during a specified event. The specified event usually consists of some critical storm, combined with critical conditions in the receiving water body.

An example of the application of receiving water response as an indicator of the effectiveness of control measures is given in a Des Moines, Iowa study [7]. This report presents the dissolved oxygen response to various treatment combinations during dry weather flow (DWF) and wet

weather flow (WWF) conditions. Wet weather flows consisted of both combined sewage overflows and separated storm runoff. The results from a receiving water model are given in Figure 11 as minimum dissolved oxygen frequency curves. In this case, 75% control of all wet weather runoff markedly improved the water quality by increasing the percentage of time during which the dissolved oxygen standard of 4 mg/L was met from 58 to 97%.

2.2.3.5 Effluent control criteria. Effluent control criteria are stated in terms of control at the point of discharge, whether controls actually occur at this point or elsewhere in the system.

Effluent control criteria can be based on prior receiving water modelling. It is implied that meeting an effluent criteria or guideline will meet a related receiving water criteria. The Des Moines study [7] showed the relationship of effluent controls to receiving response. Ontario's Water Management Policy 3, described in Section 2.2.3.1 is based on the conclusion that effluent guidelines are easier to enforce than receiving water criteria or standards.

Effluent criteria can be expressed in terms of annual loads reduction, overflow event reduction, or control of overflow for a specified event. The effluent criteria may also require that specific control measures be exercised on an area or on an effluent. Annual or seasonal discharge load reduction is based on a given percent reduction in the total mass of pollutant discharges compared to a given case, or on the total volume of a certain type of discharge. Overflow event reduction is based on reduction of the occurrence, duration, or magnitude of pollution overflow events. The interrelation of these control terms is shown in Figure 12, which compares the frequency of occurrence of overflow events and runoff volumes for a hypothetical case. The effect of various control options is also shown. Control of critical events is based on the reduction or complete control of the volume or mass load of a given critical overflow event. The efficiency of the pollution control operation or the allowable effluent concentration of pollutant material in the discharge may be specified for a critical event.

Specific measures to control erosion or retard the rate of runoff may be required in an area upstream from the point of discharge.

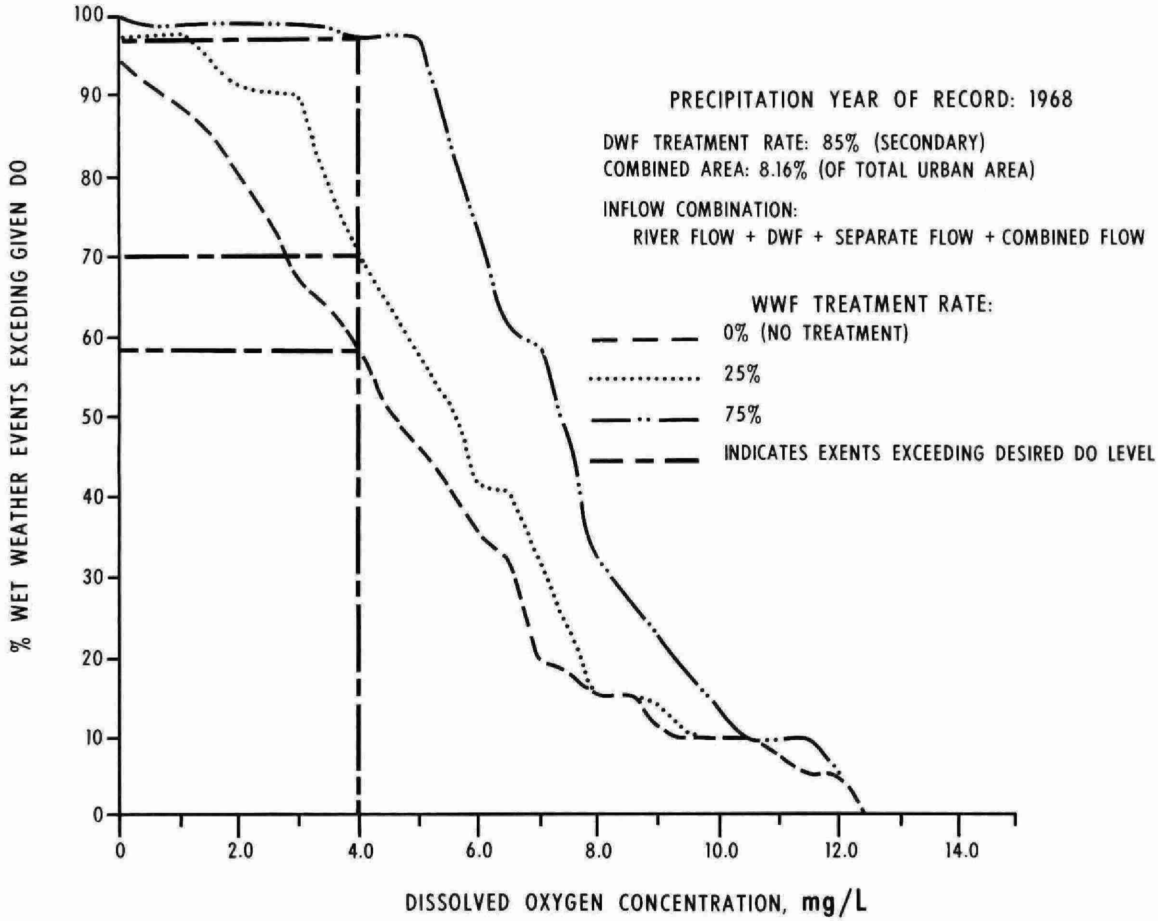


FIGURE 11. MINIMUM DO FREQUENCY CURVES FOR VARIED WWF TREATMENT [7]

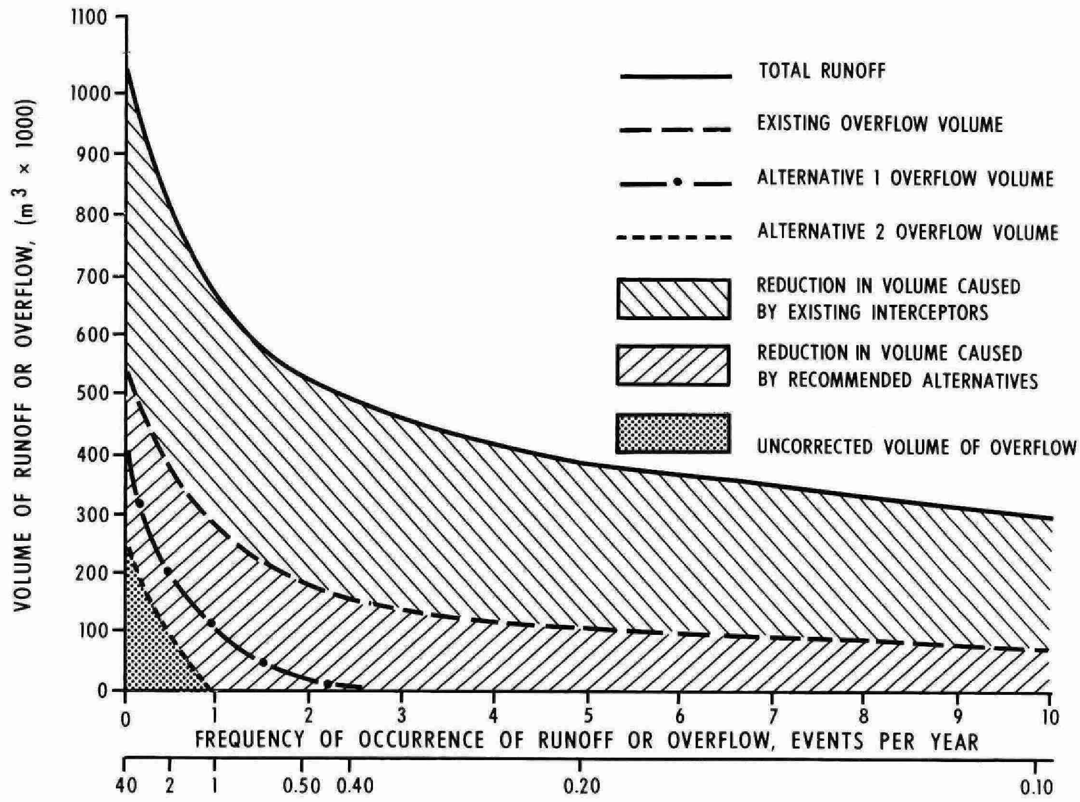


FIGURE 12. FREQUENCY OF OCCURRENCE OF RUNOFF AND OVERFLOWS [8]

2.2.3.6 Summary. In a planning framework, the defined water quality objectives are converted to specific evaluation criteria after identification of the problem and collection of information about the receiving water and the likely characteristics of the runoff pollution. The choice of receiving water response criteria or effluent criteria stated in terms of critical events or long-term performance is also dependent on these factors. The availability and expense of data collection and computer modelling also play an important role in the choice of the evaluation criteria.

2.2.4 Economic analysis

2.2.4.1 General. Decisions made by groups or organizations reflect the goals of the segments of society which they represent. These goals generally imply the maximization of something which could be called utility. Thus every organization and individual in our society is striving to maximize utility, and the definition of utility will vary according to the nature of the organization. For business organizations, it is usually assumed to be profits; for the individual, utility is often equated with satisfaction. In government agencies, utility becomes the well-being of the people or social welfare. Social welfare is the sum of all things combined for the good of a municipality, region, province, and nation. Included are such things as growth for employment, transportation, education, and quality of life which includes, but is not limited to, conservation of the environment, recreation, and aesthetics.

Unfortunately, social welfare (and environmental quality) and its maximization is a vague, ill-defined concept compared to monetary profit. However, projects can still be analysed according to their benefits and costs. Techniques of cost-benefit analysis used in environmental and flood control studies [9] are discussed in the following section.

A design criterion selects from alternative designs the one that performs "best" in terms of given objectives. The device for ordering the set of alternatives is called the ranking or objective function, and plays a key role in the search for optional designs. This ranking function must be able to be maximized in order to determine the best alternative [10]. Ranking functions often used in water resources studies are based on benefit-cost ratio, net benefits, minimum cost, and maximum benefit.

2.2.4.2 Benefit-cost ratio (B/C) and net benefit. Benefits and cost can be compared by calculating the benefit-cost ratio or the net benefits. The ratio is calculated by dividing the direct dollar benefits of a project by the costs of undertaking and completing the project. Net benefits are simply the benefits minus the costs. The B/C ratio and net benefit calculation is a quick, simple means of assessing economic feasibility. If the ratio is 1:1 or greater, or the net benefit is zero or greater, then the project or program is feasible economically. This is the minimum requirement for economic efficiency.

The B/C ratio or net benefit can also be used to assist in ranking alternative projects or programs in terms of their rate of return or net benefit to society.

The following example describes the use of the B/C ratio for flood control dams on the Thames River [1]. The Thamesford dam, when constructed at a cost of \$5 million (1975), would eliminate about 75% of the \$1.5 million average annual flood damage downstream. This annual benefit of \$1.116 million, discounted at 7% over 50 years, represents a present value of \$10.3 million in 1975. The benefit-cost ratio is thus:

$$B/C = 10.3/5.0 = 2.1$$

and the net benefit is:

$$\text{Net Benefit} = 10.3 - 5.0 = \$5.3 \text{ million}$$

The government would thus be justified in building the dam provided other tangible or intangible costs did not counter this conclusion. If a decision-maker chose not to build this dam on the basis of his assessment of the intangible costs (and benefits) then it would be presumed that the intangible costs were equal to or greater than the net benefit of \$5.3 million.

Methods are available to aid consideration of intangibles in benefit-cost analysis [11].

2.2.4.3 Minimum cost. This technique of analysis is most useful when the benefits are fixed. In many cases, the benefit cannot be quantified in monetary terms and may be represented by an environmental standard or various levels of control. The economic analysis then is directed to

finding the minimum cost of achieving a given standard or level of control. Projects which achieve the same level of control or standard can be ranked in the order of their cost and the "best" option is the one with least cost.

2.2.4.4 Maximum benefit. Often agencies have a fixed budget to assign to a project. The problem then is to determine what project or program will achieve the greatest benefit for a fixed cost. This can be carried out by analysing the cost and effectiveness of each project and ordering alternatives according to their effectiveness for a given expenditure.

All these techniques require quantification of the input costs and effectiveness of various projects. The objectives of the agency must be stated in terms that compare with the effectiveness measure used in the quantification, so that the projects can be evaluated and ranked. The goal of all these techniques is the achievement of economic efficiency and the identification of "cost-effective" solutions.

A recent report [12] presents additional uses of economic analysis and outlines techniques for determining cost-effectiveness in the following urban drainage problem situations:

- determination of optimal combinations of storage (for flow equalization) and treatment for runoff control;
- determination of optimum proportioning of funds for control of combined sewers or storm sewers in a given area, based on the costs of control and effectiveness of expenditures on each type of sewer;
- determination of optimum combinations of primary and secondary treatment devices for runoff control;
- determination of the optimum mix of wet weather control and tertiary treatment of dry weather flow for control of pollution from an entire urban area.

2.3 Planning Procedures

2.3.1 Systems analysis in planning

Systems analysis provides a formalized problem-solving framework for the planning process. The procedure, as shown in Figure 13, follows a sequence of activities from problem definition, formulation of decision

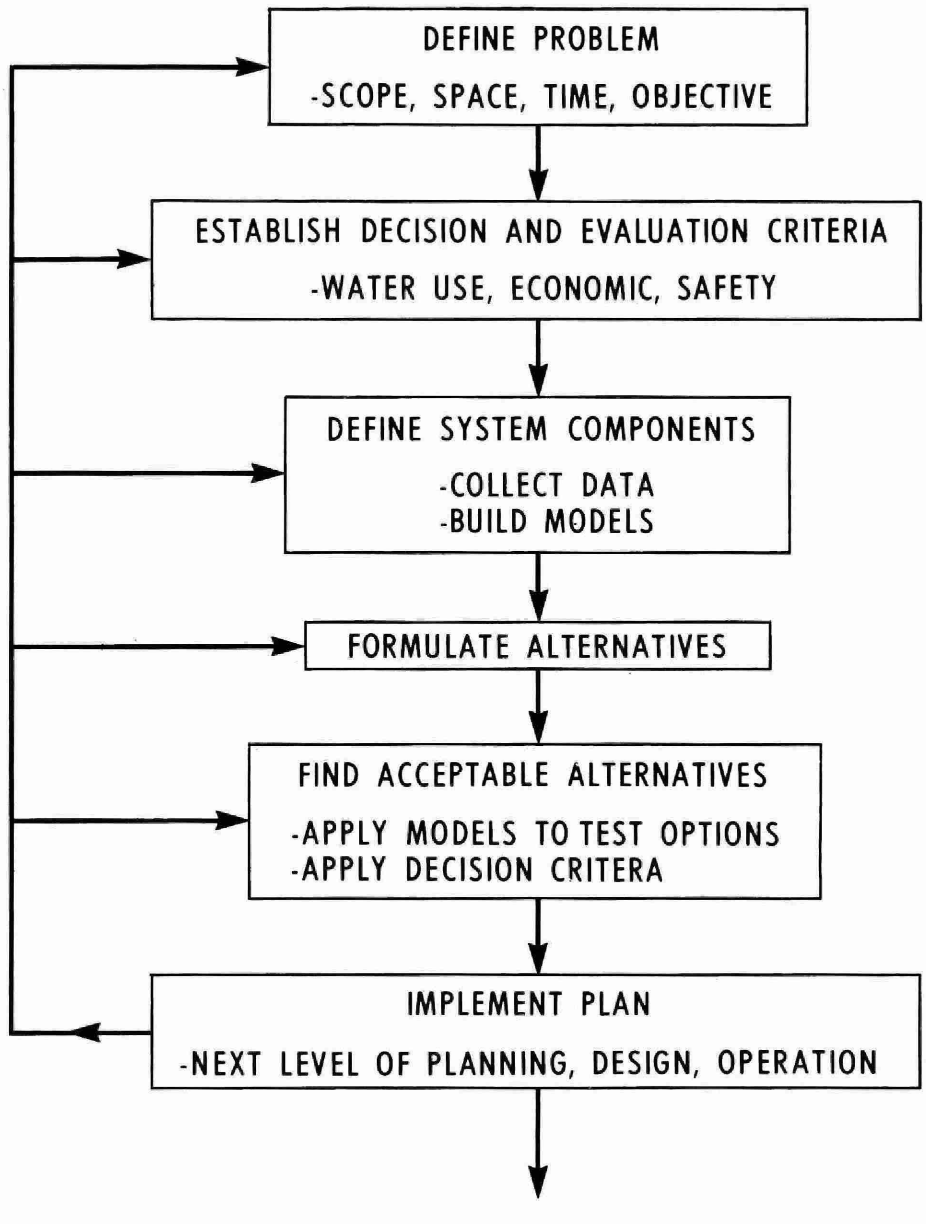


FIGURE 13. PLANNING STRATEGY - SYSTEMS APPROACH

criteria, definition of system components, formulation and testing of alternatives, and implementation of a plan of action. The approach can be used in any phase of a study from land use planning to detailed design. The procedure allows for feedback which can result in a restatement of goals, new decision criteria, redefinition of system components, and new alternatives, until a satisfactory plan of action results.

Table 4 illustrates the systems analysis approach applied to the design of a storm drainage network for a developing subdivision, and also illustrates a common shortcoming in urban drainage design. The objective of the design is narrow and constrained, and ignores site erosion problems and downstream effects. If the objective of rapid removal of water from the site was justified after consideration of potential downstream erosion, and flooding and water quality problems, then the analysis would appear to be adequate. If this same limited objective was chosen on the basis of standard practice, ignoring other considerations, then the procedure is open to criticism.

Presently most urban drainage systems are designed on a fragmentary, goal-oriented basis. Municipalities and developers are often concerned only with local criteria and the design of facilities to handle storms with two to ten-year return periods. In current terminology, storm sewers for these return frequencies are referred to as "minor system" drainage. On the other hand, conservation authorities consider flood control for streams on the basis of the regional flood which may have a return frequency of once in a hundred years or more. Such concerns are generally referred to as "major drainage". In general, the minor system is the pipe network, while the major system is the route taken by runoff when a large storm event exceeds the capacity of the minor system. A complete discussion of major and minor systems is found in Chapter 4.

It is gradually being recognized that the most cost-effective means of providing urban drainage facilities for a new area is to work from an urban drainage master plan - with consideration given to integrated planning of the major and minor systems and the downstream effects of urbanization on runoff quantity and water quality.

TABLE 4. COMMON PROCEDURE FOR DESIGN OF A STORM DRAINAGE NETWORK

Problem Definition: To design convenience pipe drainage network for a subdivision with a given road network and land use pattern, considering maximum convenience as the prime objective.

Decision Criteria:

a) Drain and route, under free flow conditions, the peak flows for a five-year design storm of appropriate intensity and duration.

b) Minimize costs subject to the foregoing and other design criteria for the municipality.

Define System Components:

a) inputs - design storm, unit costs.

b) elements - land use, road network, slope, depth, size, location, drainage element, drainage outlet.

c) output - peak flow rate for design storm, cost.

Note: The technique used in this activity is a combination of the Rational Method for calculation of peak flows, pipe sizing to pass the peak flow rate, and cost analysis for the pipes so designed.

Formulate Alternatives: Different pipe configurations.

Note: No major options are considered in this analysis, largely because of the limited scope and single objective of the problem definition given to the design engineer.

Find Acceptable Alternative: Choose pipe configuration with least cost.

Implement Plan: Construct storm drainage system.

2.3.2 Master plan, total management and relief studies

In this section, prepared by J.F. MacLaren Ltd., under contract to the Urban Drainage Manual Committee, the systems approach is applied to three typical urban drainage studies. In each case the approach involves a systematic analysis of alternatives based on well-defined objectives, but the scale and objectives of the three studies vary considerably, as shown in Figure 14. The studies are a "Master Drainage Plan" for a developing area, a "Total Environmental Management Plan" for an urban area, and a "Flood Relief" study within an existing urban area. In the following discussion only the development of the master drainage plan is presented in detail; the total environmental management study and the flood relief study are summarized.

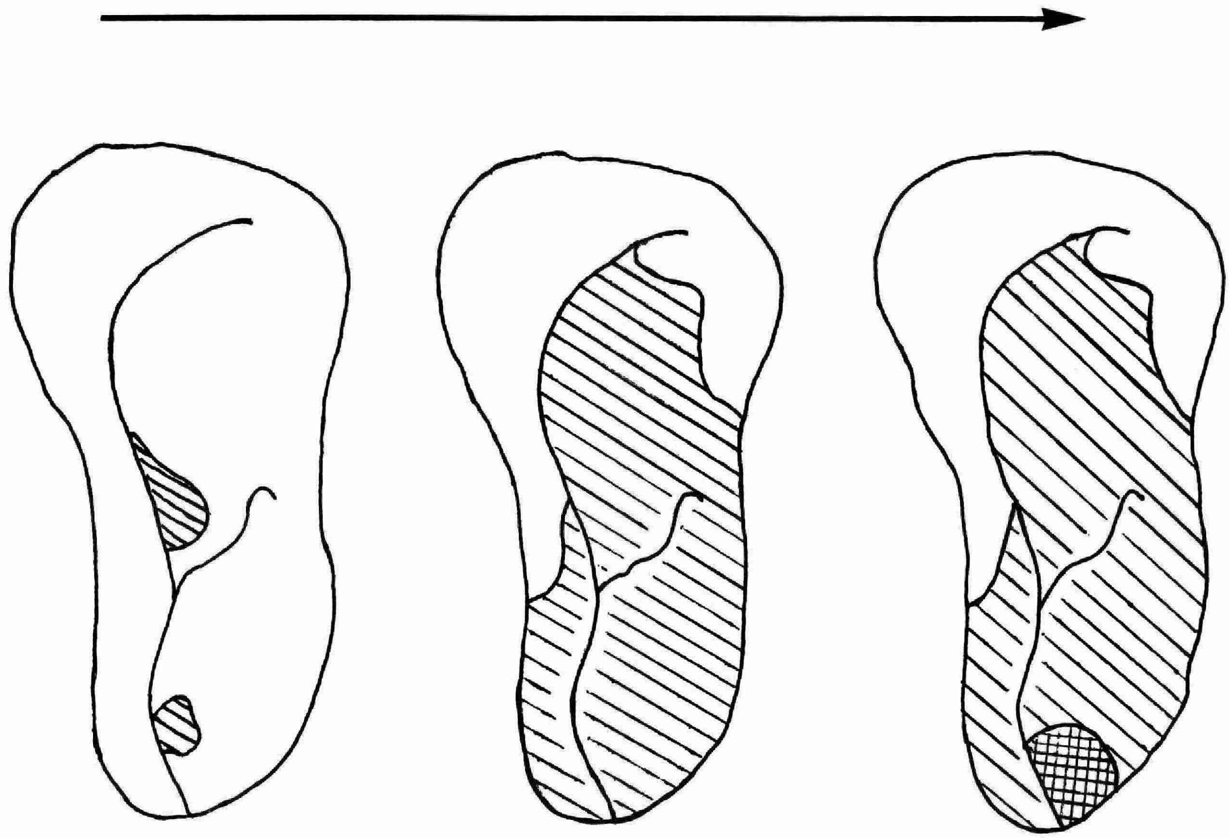
2.3.2.1 Master drainage plan. A master drainage plan is usually required for a proposed development in a largely rural watershed, particularly when the scale of development is such that a significant impact on the natural drainage system may result. The analysis of alternative drainage systems is necessarily at a conceptual level only since the details of the development will probably not be finalized at this stage. While the major emphasis is on post-urbanization flow quantities, the analysis should include an awareness of the desired water uses for the urban condition.

An important aspect of master drainage planning is consideration of the whole system, including the interrelationship of the so-called "minor" and "major" urban drainage systems. Broadly speaking, the differences between minor and major drainage systems are the differences between drainage and flood control or between convenience and damage prevention systems. This difference in function results in different potential benefits. Benefits of the minor system are largely intangible, while the major drainage system can provide important flood damage protection. Table 5 compares possible benefits to be derived from properly planned minor and major drainage systems.

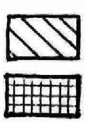
Master drainage planning is generally carried out by considering various alternatives for the major drainage system. Components of the major system may include some trunk sewers, open channels, main culverts and highway bridges, natural streams, and main ponding facilities. Although diffuse storage in the minor system is useful for reducing storm

INCREASING SOPHISTICATION AND COMPLEXITY

OF STUDY



DEVELOPING AREA	EXISTING URBAN AREA	EXISTING URBAN AREA
-Master Drainage Plan Study -Study area is entire watershed	-Total Environmental Management Study -Study area is entire city (or watershed)	-Flood Relief Study -Study area is a portion of the city



Urbanized area

Portion of city to be studied

FIGURE 14. COMPARISON OF MASTER DRAINAGE PLAN, TOTAL ENVIRONMENTAL MANAGEMENT, AND FLOOD RELIEF STUDIES

TABLE 5. POSSIBLE BENEFITS OF PROVIDING DRAINAGE FOR MINOR AND MAJOR FLOWS

Minor Flows	Major Flows
Reduced Street Maintenance	Damage and Liability Reduction
Reduced Traffic Delays	Land Value Enhancement
Improved Conveyance	Protection of Life
Improved Aesthetics	Improved Aesthetics
Alleviation of Health Hazards	

water peak flow rates and improving storm water quality, the effect of such storage should be neglected (for safety purposes) when deriving flows for the major system.

The requirements for minor or convenience drainage can then be defined in relation to the major system. At the planning level it is usually possible only to consider the trunk sewers of the minor system, with the laterals and collectors considered at a later stage when detailed development plans and land use patterns are available. However, general proposed land use patterns must be evaluated in order to allow for suitable trunk sewer outfall points to the major system.

2.3.2.2 Activities in master drainage planning. Figure 15 outlines the main activities of the master drainage planning process and their inter-relationships. The diagram is generalized and specific areas and plans for development may require modifications to various activities. Bracketed numbers {} in the following text refer to individual activities shown on Figure 15.

The first stage in the planning process should involve meeting with representatives of relevant agencies in order to more closely define the specific area of concern, the planning and drainage objectives, and the criteria which are to be involved as a measure of the effectiveness of the drainage alternatives. Since responsibility for drainage, flood control, and environmental concerns rests with different agencies, this first activity is an important part of the comprehensive planning process.

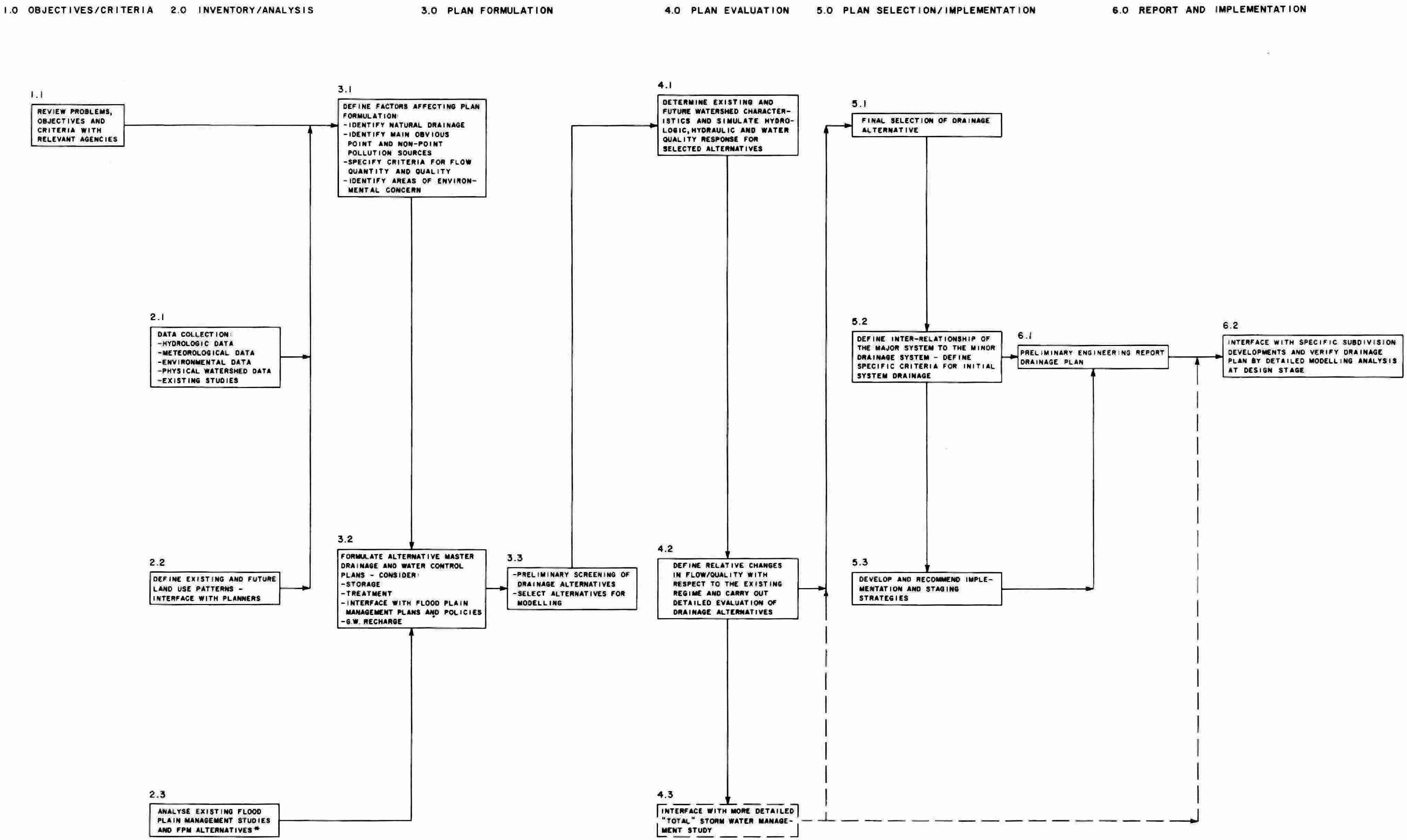


FIGURE 15. MASTER DRAINAGE PLANNING STUDY - ACTIVITY DIAGRAM

Where policies have already been specified it may only be necessary to alert the agencies concerned that studies are underway and their eventual reviews and approvals will be required.

The next main set of activities {2.0} comprises the collection and analysis of existing data. The data collection phase {2.1} involves review and analysis of existing studies and also includes collection and preliminary analysis of meteorological, hydrologic, and water quality data. Preliminary data analysis could involve application of tools such as the DAM (Data Analysis Model) developed during a recent storm water management study [14]. In some situations review of local ecological problems may also be required.

It is necessary during the inventory stage to define existing and future land use conditions {2.2} in order to assess potential changes to the hydrologic regime (during stage 4). Future land use patterns can be determined by examining local official plans or, where official plans do not exist, by involving local planners in the drainage planning process. In some instances, it may be necessary later to revise proposed land use patterns to accommodate the "best" master drainage alternative. For example, the water quality impact on the receiving water body may be a factor in the assessment of alternative land use plans. Land use planning is an important factor in consideration of flood control alternatives. It is, therefore, necessary to consult the local conservation authority to determine the current flood plain management policies. Flood plain management alternatives can significantly affect the master drainage planning process. For example, in areas where non-structural flood plain management policies are favoured over structural techniques, it may be more difficult to provide adequate protection against stream erosion due to urban-related increases in storm flows.

In developing alternative plans {3.0} it is first necessary to define the main factors affecting plan formulation. It is also necessary to investigate the natural drainage conditions in order to identify specific drainage problem areas. In modern drainage practice it is becoming more common to utilize natural water courses as part of the major system with as few changes as possible. Natural water ways provide on-stream storage of storm water flows as well as environmental and aesthetic benefits.

By identifying the main obvious point and non-point pollutant sources and amounts, it is possible to identify specific areas of environmental concern {3.1} (e.g., minimum flows, vegetation, wildlife), and to identify the relative importance of such aspects as, for example, water quality considerations.

The formulation of alternative drainage plans should examine the possible effects of storm water runoff on the receiving water body and consider storage and possible treatment of these flows {3.2}. This step should also include consultation with conservation authorities to determine and integrate local flood plain management policies on flood levels and erosion control with the master drainage plan.

Preliminary screening of drainage alternatives {3.3} is then carried out to identify those alternatives for which more detailed modelling is desirable. Land use restrictions and requirements and environmental, as well as functional, concerns should be considered in this preliminary screening. The preliminary screening could also involve application of a "coarse" simulation technique such as the STORM model described in the next chapter. Such a model can provide a preliminary assessment of the effect of suggested storage and treatment options on the quantity and quality of storm water runoff.

Once two or three primary alternatives have been identified, a more detailed evaluation of these alternatives can be carried out {4.0}. This should involve the application of more sophisticated simulation models, such as the EPA-SWMM also described in the next chapter. If changes in low flows are also of concern, then application of a more flexible continuous simulation model may also be considered at this stage {4.1}. Application of these simulation models for existing and future watershed conditions can provide a great deal of insight into the hydrologic, hydraulic, and water quality response of the receiving water to selected alternatives.

In cases where only a relative assessment of the alternatives is required it may not be necessary to calibrate and validate the simulation model. However, where the absolute value of flows and water quality parameters may be important in selection of the alternative plan, calibration of the models to existing conditions may be necessary prior to

simulating the primary alternatives. The EPA-SWMM has been validated for several Canadian urbanized watersheds. The mathematical simulations of storm water flows, water levels, and water quality parameters allow an assessment to be made of relative changes in these parameters for existing and future drainage conditions.

The next stage of the investigations is a comparison of drainage alternatives {4.2}. A matrix scoring method of assessment has been adopted in some studies in order to compare noncommensurate benefits such as costs and environmental factors. Such methods derive a "score" for each alternative, providing very useful comparative information for decision-makers.

Upon final selection of the master drainage plan {5.1}, it is necessary to formulate general guidelines defining the interrelationship of the major and minor system. For example, minor systems are usually designed for 2-10 year frequencies based upon local selection of a level of protection against nuisance drainage problems. However, the effects of the major system on minor system drainage should also be considered, for example, the possibility of backwater from some components of the major system such as storage reservoirs, etc. Also, while a trunk sewer in the minor system may only carry the 2-10 year flow, the associated surface street drainage system may be designed to carry the 100 year flow without causing significant damage.

Implementation {5.3} of the recommended plan is the most difficult phase of the urban drainage planning process. Political, financial, development staging, and jurisdictional problems must all be considered. Based on the anticipated staging of development, a corresponding schedule of implementation should be prepared, taking into account the hydrologic and environmental problems identified during the analysis stage. The finalized plan and staging recommendations should be incorporated in a preliminary engineering report {6.1}. At the design stage {6.2}, drainage services for individual developments must be considered and detailed modelling should then be carried out to include examination of the effect of laterals, on-site storage, etc., in relation to the requirements of the master plan.

2.3.2.3 Total environmental management study. A total environmental management study can be applied to an existing development as a means of determining the most cost-effective way of achieving defined environmental objectives. Consequently, total management involves an analysis of all sources of water pollution over the entire study area (usually at a city-wide scale). Alternative measures for meeting the environmental objectives are tested and recommendations made. The St. Thomas demonstration project discussed later in this chapter is an example of a total environmental management study.

A total management study is useful in developing a plan for reduction of pollutant discharges in conjunction with a drainage plan for both existing and future drainage conditions. Such a study may be coordinated by either the local municipality or by a provincial agency. The main study objective would be to define the optimum mixture of controls required for point and non-point pollutant sources in order to meet existing or proposed quality criteria in the receiving water body.

The general approach employed in the total management study is similar to that discussed in detail for the master plan. The main activities in a typical total management study are illustrated in Figure 16.

2.3.2.4 Flood relief study. Continually changing urban land use patterns may make existing drainage systems obsolete because of increases in the amount of impervious area resulting from continued development or revisions to drainage standards, etc. As development continues in urban watersheds, existing combined and separated storm sewers become overloaded due to increased storm flows from upstream drainage areas. Typical problems caused by overloading include:

i) basement flooding resulting from back-up of combined sewers;
ii) basement/street flooding resulting from back-up of separated storm sewers;
iii) storm overflows to receiving water bodies from combined sewer systems.

A relief study is designed to solve these problems. Generally, the major emphasis of the study will be placed on (i) and (ii). The scope of the environmental analysis will be dictated by the total management

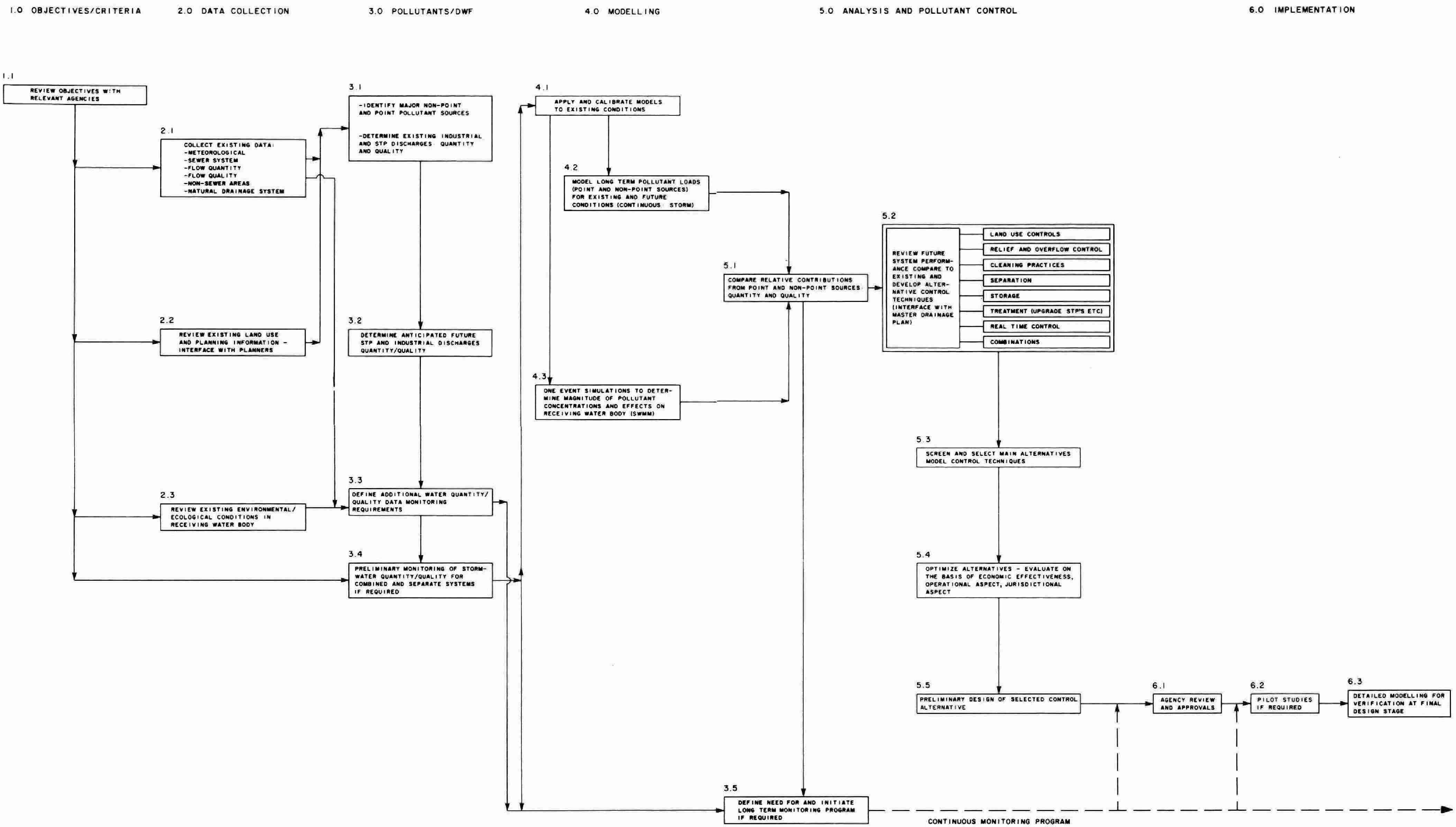


FIGURE 16. TOTAL ENVIRONMENTAL MANAGEMENT PLANNING STUDY - ACTIVITY DIAGRAM

study. Again, the systems approach discussed earlier is applied. The main activities in a typical relief study are illustrated in Figure 17.

2.4 Application of Planning Principles

The logical planning sequence would be to carry out river basin and regional studies first and to establish local objectives within the context of these larger scale units. However, it is clear that the resources of the Province cannot provide detailed river basin or regional plans everywhere they are required. Also new urban development will not await completion of large scale planning studies, nor should remedial flood, erosion, and pollution control programs be halted.

The need for basin and regional planning should be recognized at all levels of government and action taken to implement these programs in areas of greatest need. In the absence of basin or regional plans, planning studies for smaller units of land or for local remedial measures should take account of wider objectives, and should be directed towards the ultimate goal of developing basin plans.

The following sections discuss the application of the urban drainage concepts and procedures discussed previously to the planning of new urban development and to control problems in existing urban areas. It is recognized that regional or basin-wide studies will encompass both types of planning studies; however, it is felt that the categorization below will emphasize a desired approach and be of use to planners today. Various aspects of the implementation of this coordinated planning approach are discussed in Chapter 6.

2.4.1 Planning for new urban development

The following considerations should be incorporated in planning for new developments.

i) Regional and watershed planning considerations should be incorporated in drainage planning for new developments. This can best be implemented through the development of watershed plans by interdisciplinary groups, coordinated by regional governments or conservation authorities. Drainage planning at all levels should take cognizance of regional and watershed considerations by adhering to the results of watershed studies. Drainage plans

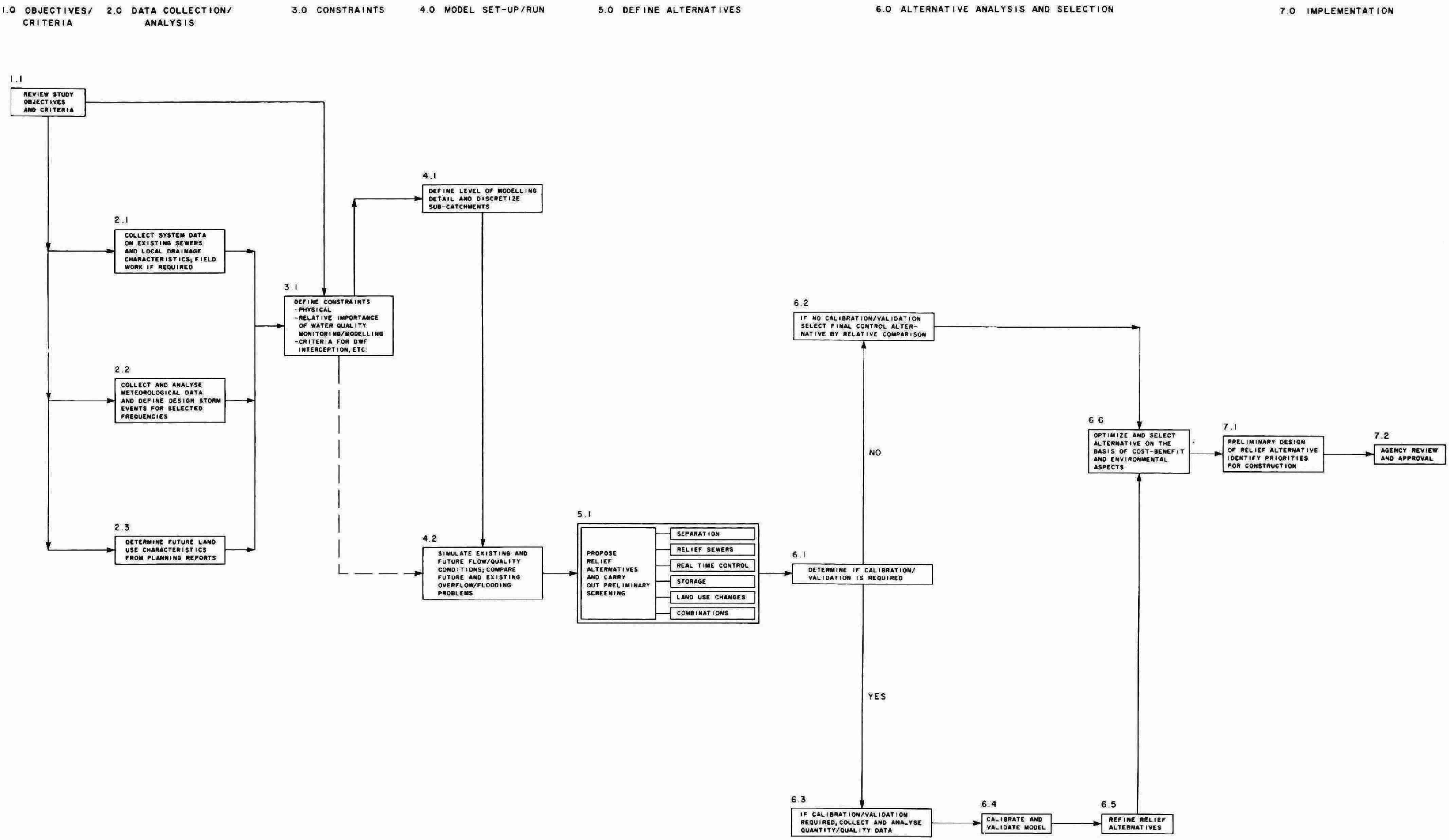


FIGURE 17. TYPICAL FLOOD RELIEF STUDY - ACTIVITY DIAGRAM

should add to the knowledge and data base required to develop or improve regional and watershed plans. Drainage planning studies at all levels should be broadened in scope and objectives to give proper consideration and balance to environmental and flood control concerns and to allow consideration of numerous alternative means of meeting the objectives.

ii) Land use planning for urban development should include drainage considerations at early stages to incorporate drainage objectives which may significantly affect the land use type, density, and location. Opportunities for innovative drainage, and environmental and erosion control will occur at all stages of planning and, if taken advantage of, will result in better land use plans.

iii) Erosion and sedimentation problems should be considered in the first stage of site planning for new development by identifying "hazard lands", such as ravine slopes, which are very susceptible to erosion when the stabilizing effect of vegetation is removed. These erosion-prone areas should be considered for inclusion in open space (parkland) designations to minimize the effects of erosion and consequent sedimentation through avoidance. Later stages of planning and design should consider the effect of various development options on erosion, as well as the site characteristics such as erodible soils, slope, water volume and velocity over the area, and the sensitivity of the downstream area to damage from sediment. Remaining erosion problems should be controlled by specific measures on the construction site as discussed in subsequent chapters of this report.

iv) Flood plains should be outlined and development forbidden except in certain circumstances. For example, flood plain development would proceed in rare cases if it is in the public interest, if structures can be adequately flood-proofed and if there is no detrimental effect upstream or downstream from this use, or if the detrimental effects can be alleviated.

Appropriate flood control for every residential structure should be provided through a combination of drainage works, grading, and

relief easements for overland flow. The major and minor systems should be considered. Additional design criteria should be adopted for protection of roads.

v) The effects of urbanization on changes in the hydrologic regime (such as the peak flow) should be minimized, especially in cases where downstream flooding or erosion would be aggravated. In cases where such problems already exist, it would be reasonable to adopt an objective of no increase in the peak rate of runoff. The design storm frequency to be chosen should be based on the channel geometry, erodability, and potential for flood damage. Control of peak runoff rates for downstream erosion protection should be based on the erosion potential of various design storms. In some cases, it is reasonable to design peak flow attenuation systems for control of frequent storms (one to five year return), since designing for less frequent storms would provide insignificant increased benefits for erosion control on an annual basis. Maintenance of base flows in streams should also be considered in planning studies. This could include protection of recharge areas from development as well as specific measures to increase recharge during the disposal of urban runoff.

vi) The pollution load of urban runoff should be reduced by a combination of techniques outlined in subsequent chapters. Appropriate levels of control depend on the receiving stream; for most developments, minimum control levels should consist of some form of sediment and erosion control. Performance criteria for the design of pollution control structures should be based on the average annual pollutant removal efficiency for all storms in a typical year. Specific runoff events should only be used for the design of such systems when their relationship to the annual control efficiency is known or when it is necessary to protect downstream quality during the specified design storm.

vii) Facilities which have a multi-objective use should receive a high priority for consideration in new developments. The prime example of this is the use of multi-purpose ponds for such diverse functions as:

- erosion control during construction;
- sedimentation on a long-term basis;
- attenuation of peak rate of runoff for downstream erosion and flood control;
- cost reduction by reducing the cost of downstream conduits or outlet works;
- recreation and aesthetics;
- maintenance of natural infiltration, water table levels, and base flows in streams.

2.4.2 Planning for remedying runoff problems in urban areas

The following measures and methods are recommended for use in the environmental and flood control planning process.

When carrying out improvements to or expansion of existing sewage treatment facilities, appropriate environmental and flood control objectives should be included to ensure that the whole environment benefits, all costs are considered, and an acceptable level of flood control is provided.

As a minimum requirement, combined sewer overflows should be controlled (reduced and/or treated) to alleviate the health hazards and severe pollution which result from the discharge of untreated sanitary wastes. The appropriate level of control or treatment should be based on considerations of the local receiving water body as well as the effect on the ultimate receiving water body, i.e, the Great Lakes. The principle of economic efficiency should be incorporated when considering expenditures for combined sewer control. The level of control chosen should be dependent on the receiving water response as well as the costs of controlling separated storm runoff and/or higher levels of treatment of sanitary wastes.

The scale of planning for environmental control should be based on metropolitan regions and river basins.

Runoff pollution should be reduced through a combination of source controls (see Chapter 4).

Basement flooding in combined storm and sanitary systems should be halted. The level of protection should be of the same order as that for flood plain protection in view of the potential for property damage and health hazard.

Every municipality should carry out a total environmental management study prior to the next expenditure for alterations to the sewage treatment plant, particularly if an upgrading of the facility to a higher level of sanitary waste treatment is proposed. This is the logical time to assess the most beneficial options and these can only be evaluated if the pollution load from the entire urban areas is considered. The total management study should be carried out within the framework of the approach described in Section 2.3, utilizing the measures outlined above.

2.5 Planning Examples

The concepts and principles of planning presented in earlier sections are best illustrated by examples of their application.

The Thames River basin study in southwestern Ontario is presented in Section 2.5.1 as an example of river basin planning which includes consideration of both flood and water quality control objectives as well as the use of least-cost analysis.

The various levels of environmental planning available in the United States as part of recent legislation are presented in Section 2.5.2. Planning studies are of three types - river basin studies, area-wide waste management studies and facilities planning studies.

A demonstration project undertaken in St. Thomas, Ontario is described in Section 2.5.3. This study illustrates the use of advanced storm water management techniques to develop solutions which consider multiple objectives for environmental and flood control.

The experience of the City of Mississauga in developing a storm water management policy is described in Section 2.5.4. The requirement for individual consideration of each watershed is highlighted by this example.

Section 2.5.5 describes the Hanlon Creek study - an example of an environmental study of a watershed in the process of urbanization. Several of the study recommendations regarding environmental protection and maintenance of the natural hydrology have been adopted in subsequent construction efforts.

An evaluation of the effects of proposed urban development on watercourses in the City of Oshawa is described in Section 2.5.6.

Storm water management considerations for the proposed Thornhill-Vaughn community in the Town of Vaughn are outlined in a recent report on

servicing schemes for the area [21]. The proposed storm water servicing methods include consideration of the major and minor drainage systems and provisions for erosion control and for retarding the rate of runoff. These measures are designed to protect downstream natural and man-made drainage systems from flood damage, erosion, and water quality degradation.

In April, 1977, the City of Burlington adopted new drainage policies and introduced newly-developed storm water management techniques [30]. The policy of the city is in general agreement with the concepts outlined in this chapter. The policy is being applied to a developing watershed where flooding of downstream areas will result if upstream development causes increased runoff [31].

The Town of Oakville [34] and the City of Kitchener [35] have also adopted drainage policies and criteria consistent with the concepts presented in this report. The independent acceptance of these concepts, without pressure from senior governments, implies that the concepts are both practical and implementable at the municipal level.

Studies on the effects of urbanization on hydrology and water quality have been carried out for several proposed urban developments in North Pickering [22], the Regional Municipality of Ottawa-Carleton [23,24], Townsend community [25] and for Shriner's Creek in the Regional Municipality of Niagara [26].

2.5.1 Thames River study

The Thames River study [1] is a recent example of comprehensive watershed planning in which both flood control and water quality objectives were considered.

The Thames River supports a growing urban population and an active agricultural community. This has resulted in water quality degradation affecting beneficial uses such as water supply, recreation, and fishery. Flooding of urban areas and agricultural lands is also a problem.

The overall objective of the study was "to develop guidelines for water management planning in the Thames River basin which would ensure that an adequate quantity of water at a satisfactory quality is provided

for recognized water uses in the river basin at the lowest cost, and that flood and erosion protection is provided consistent with appropriate benefit-cost criteria" [1].

In order to resolve the existing problems effectively, water management objectives were developed and alternative courses of action were evaluated. The short-term water quality objective was defined to be the maintenance of existing water quality where it is satisfactory for fish and aquatic life and recreation, and to improve quality to this level where it is presently degraded. The long-term objective is to upgrade water quality as much as possible to enhance conditions for fish and aquatic life, as well as to maximize other beneficial water uses. Dissolved oxygen criteria and other specific water quality criteria were developed to meet this objective.

Flood control projects were based on providing a combination of flood plain management and major control structures such as dams and dykes. These were designed to provide control to the point where the marginal costs of increased control begin to exceed the benefits provided.

Techniques used in the evaluation process included a public consultation program, water use studies, a dynamic water quality simulation model to assess specific options with respect to dissolved oxygen criteria, and a flood routing model to assess flood control benefits.

It was concluded that flood control in the basin would require the construction of one or more large dams, and a detailed flood control benefit-cost analysis of proposed major dams was carried out. In addition, as flood control and water quality improvement options are closely interrelated, various combinations of the proposed reservoirs and waste management options were examined in a systems context.

The least-cost ordering of systems options was utilized to identify attractive options, and these were then subjected to a further review of unquantified benefits and costs in order to develop a proposed course of action. This course of action included recommendations for control of point source discharges as well as nonpoint discharges from urban and rural sources. The combination of multiple-use reservoirs which provided the highest benefit in terms of flood control and environmental quality was identified.

2.5.2 Planning studies in the United States

Recent legislation in the United States (Federal Water Pollution Control Act Amendments, 1972, Public Law 92-500) established three basic environmental planning programs [16]. These overlapping levels of planning are basin planning, area-wide waste treatment management planning, and facilities planning.

A basin plan, provided for under Section 303 of the Act, is the water quality management plan for all the land and surface water area in a designated river basin. Its purpose is to provide the state with the framework for making coordinated and effective water quality decisions on a river basin scale. The basin plan serves this purpose by identifying the basin's water quality problems and by outlining necessary corrective measures. In delineating these problems, the plan must determine existing water quality, recommend necessary revisions of the state's water quality standards, and identify significant points of pollution. In prescribing the corrective measures, the plan must determine effluent limitations and control strategies for point sources, identify the need for facilities planning and for area-wide waste treatment management planning, and set priorities for construction.

Section 208 of the Act provides for the designation of urban industrial areas that have substantial water pollution problems. Once designated by the state governor, the appropriate regional/local planning agency became eligible to apply to the Environment Protection Agency (EPA) for funding to undertake area-wide waste treatment management planning. The area-wide waste treatment management planning process is one of the keys to proper planning for areas that have water quality problems caused by both point and non-point sources. The plan is comprehensive, identifying the needed technical pollution control measures and also providing the means to implement them. The pollution abatement controls utilize a combination of land use measures and management and regulatory programs as well as structural methods.

The third type of planning required by Sections 201, 204, and 212 of the Act is facilities planning. A facilities plan is a detailed plan leading to the construction of municipal sewage treatment facilities. The plan will generally cover an area smaller than the 303 plan, or the 208

plan, but large enough to permit cost-effective planning in a prescribed area. A facilities plan will include many of the items considered in area-wide waste treatment management plans, but will address these issues as they specifically relate to a proposed project. The plan is designed to determine the most cost-effective means of providing municipal waste treatment for an area. An environmental assessment of the plan and an economic assessment of various alternatives are included in the cost-effectiveness decisions. These alternatives would address land treatment, reuse of wastewater, flow reduction measures including correction of excessive infiltration, the treatment of overflows, alternate system configurations, phased development of facilities, and improvement of operation and maintenance in existing facilities in the area.

2.5.3 St. Thomas storm water management demonstration study

A study sponsored by the Canada Mortgage and Housing Corporation was initiated in October 1976 in St. Thomas, Ontario, to demonstrate techniques for controlling urban runoff problems for the entire city [17]. St. Thomas is a small city (1975 population 26 853) with a large proportion of the urbanized area served by combined sewers, which results in recurrent basement flooding problems, combined sewer overflows, and sewage treatment plant disruptions in wet weather.

Kettle Creek, which receives storm and combined sewer runoff and the effluent from the sewage treatment plant, has a severely limited capacity to assimilate the pollutants in the discharges. Plans to separate the combined system have been partially implemented, with a projected net cost (to the city) of $9.5 million in the period 1970 to 1980. It was hoped that this project would provide less costly alternatives than complete sewer separation.

The purpose of the study was to apply advanced storm water management techniques to find cost-effective solutions to storm water problems. The solutions were required to meet both objectives of minimizing pollution loads to Kettle Creek from storm water, combined sewer overflows and treatment plant by-passes, and minimizing health hazards and property damage from the flooding of basements and other areas during storm events. As well, the study report presents a generalized methodology suitable for application in other cities with similar problems.

2.5.4 Storm water management in the City of Mississauga

The City of Mississauga is undergoing rapid urbanization due to development pressures from nearby Metropolitan Toronto. The city borders Lake Ontario and is drained by 30 watersheds ranging in size from 160 - 110 000 ha (400 - 264 320 acres). The lower portion of the city is developed. The sewers and smaller creeks in this area are under or at capacity, and as urbanization of the upstream land continues, flows are expected to increase. Existing severe erosion problems will be aggravated by the increased rates of runoff.

Recognition of these problems led to an internal study [18] of storm water management with the specific objective of reducing runoff in all new developments. Watersheds were first classified according to their shape, location of existing urbanized area, and access to a receiving body. A receiving body was considered to be a major river or lake on which increases in flow from urbanization would have no appreciable effect.

The study recommended that, in general, watersheds with undeveloped areas in their head waters apply detention techniques to control future increased runoff, while watersheds with undeveloped areas close to a receiving body continue to use conventional techniques. This is in recognition of the timing of peak flows in the watercourse. Thus, runoff from new developments in upstream areas is detained until peak flows from areas near the watershed mouth have passed to the receiving water.

The study concluded that the ultimate solution to damage from large storm events lies in the construction of major structures for storing or conveying flood waters or restriction of development in flood plains.

2.5.5 Hanlon Creek study

Hanlon Creek, a small watershed on the southern fringe of Guelph, Ontario, drains an area of 2950 ha (7300 acres). A study [19] was undertaken to determine the long-term effects of urbanization, as well as the specific effects of construction of an expressway, on the ecology of the creek. The creek has a well-sustained base flow from springs and continuous lateral inflow from groundwater recharge areas in the watershed. It is an important water source for a waterfowl park.

The study identified the sources of water, and the location and extent of groundwater recharge areas for the creek. In addition, the stream and groundwater-dependent elements of the local ecology were identified. For example, a cedar woodlot adjacent to the creek is dependent on a high water table, and the presence of organic soils. The woodlot shades the water, maintaining cool temperatures, and the associated organic soils control sediment and filter out nutrients from surface flow towards the creek. The high water table is dependent on continual recharge and sustained base flows. Any factor that disturbs this balance will alter the system and thus the water quality and use of the creek.

Various development alternatives were outlined as part of the study, with major consideration given to the potential effects on base flow and water quality in the creek. The following specific control measures were recommended during development of the watershed:

i) consideration should be given to renovating storm sewer water naturally by percolation through the soil;
ii) groundwater recharge areas should be protected from development;
iii) erosion and sediment control during construction is necessary to preserve the creek;
iv) the organic soils and cedar woodlots adjacent to the creek should not be developed.

Since the study, construction of the expressway has been completed and some areas of the watershed developed. Detention and retention ponds for erosion control and storm water renovation were built as part of the construction activity and are described in more detail in Chapter 5.

2.5.6 Storm water management in the City of Oshawa, Ontario

In 1975, the City of Oshawa retained a consultant to study the north-east sector of Oshawa, the principal objective being to investigate the effect of further upstream urban development on flood problems already experienced downstream [20].

Urban development in the city had progressed to the point that downstream drainage facilities were inadequate to cope with further runoff

increases associated with urbanization. In the city, natural watercourses act as arterial storm sewers and the lower reaches suffered both an increasing frequency of flooding events and bank erosion. Faced with the prospect of a considerable population increase and pressures from already affected residents, the city decided to undertake a study of the whole problem.

The study was divided into three interrelated elements:

- an environmental and ecological appraisal,
- urban expansion concepts,
- hydrological analysis of watercourses.

The environmental appraisal included an extensive inventory of existing features, described in terms of physical uniqueness, biological quality and diversity, aesthetics, and recreational potential. All woodlots and major fencerows were rated in terms of quality and function and recommendations were made for their potential use. An analysis of soil types provided information essential to the storm runoff analysis and erosion aspects. The drainage capability of the soils was classified and physical development constraints identified.

Superimposing major physical features (Lake Iroquois shoreline, prime woodlots) on existing or proposed development constraints (road extension, hydro easements, oil pipeline) resulted in the development of a parkway corridor concept. The corridor was seen to provide an unique opportunity for recreation, reforestation, storm water management, and urban-related uses.

The 1975 city population of 100 000 is expected to increase by as much as 150 000 in the next 20-30 years, with up to 100 000 people located in the north-east sector. Applying the city density criterion of 85 persons/ha (35 persons/acre), the north-east sector would provide accommodation for up to 85 000 people, assuming the corridor concept and reserving flood plains and areas of poorly drained soils as public open space.

This land use concept was tested for flooding and downstream erosion impact. Flood flows generated by the regional storm were calculated using a unit hydrograph computer model and the associated flood plain developed using a backwater program. Extensive flooding was predicted

and alternative methods of relieving the problem were analyzed. The recommended solution involved removal of all buildings from the flood plain, local bridge and culvert improvements, and channel improvement and diversion. The resultant flood plain would become the most significant feature of the public open space system in the community, linking nodal points such as schools and local parks with walking and biking pathways.

Urbanization was predicted to have a profound effect on runoff volumes and erosion. Erosion sensitivity was tested with the two-year return period storm. Watercourse flows and therefore velocities were calculated corresponding to both present urbanization and that proposed in the land use concept. Flows were found to increase by as much as 400 percent in some reaches. Locations and treatment for particular erosion sensitive reaches were identified.

The recommended solution to the potential flooding and erosion problems has been estimated to cost $7.1 million. The city is undertaking preliminary engineering of the high priority element of work - a stream diversion - costing approximately $840 000.

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3 CHARACTERIZATION OF FLOWS IN URBAN DRAINAGE SYSTEMS

The purpose of this chapter is to discuss quantity and quality of flows in urban drainage systems. It is recognized that flows in sewers are combinations of various types of wastewater and drainage effluents. The quantity and quality of flows in drainage systems can be derived by synthesis of all flow components and by taking into account the effects of the drainage system on the synthesized flow. Quantitative and qualitative characteristics of these flow components are discussed in Section 3.1. When identifying individual flow components, the existence of two basic types of sewer systems is recognized (see Figure 18). Older communities are served by combined sewer systems in which a single sewer carries domestic sewage as well as surface runoff (storm water). Other possible flow components include industrial and commercial wastes, infiltration and inflow into sewers, and foundation drainage.

Separated sewer systems are more recent. Domestic sewage, and industrial and commercial wastes are conveyed by sanitary sewers; surface runoff is conveyed by storm sewers.

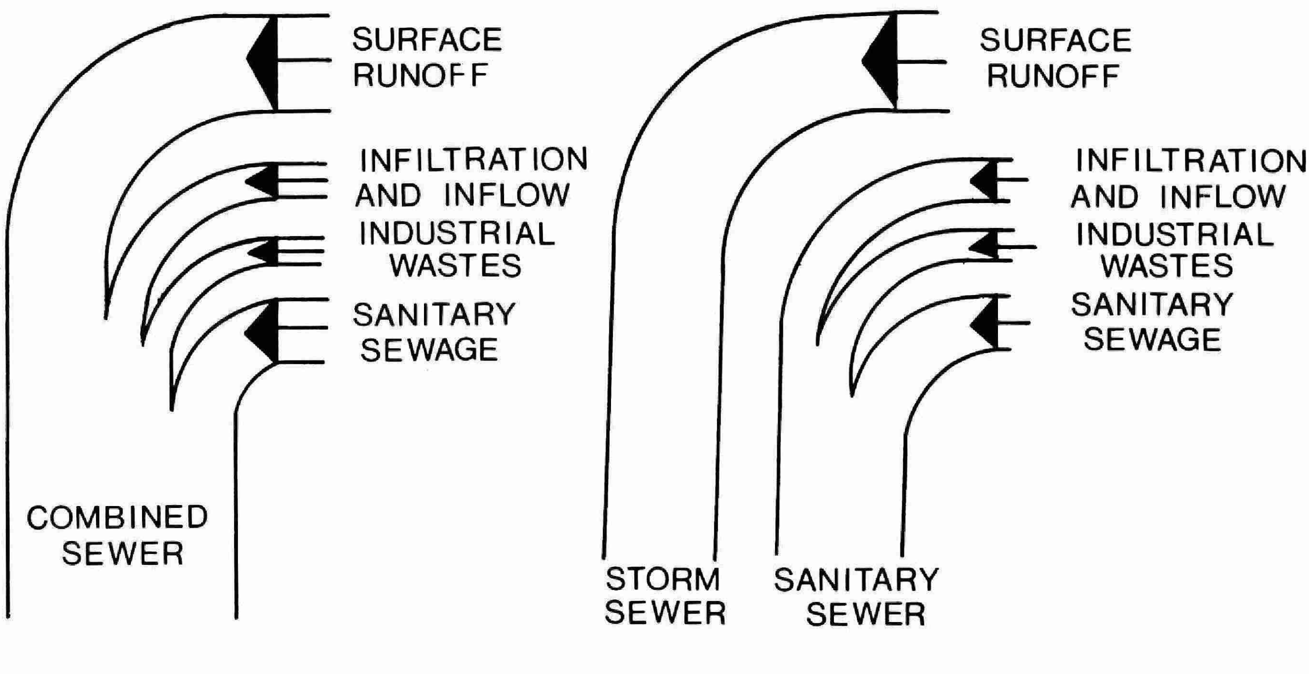

FIGURE 18. COMBINED AND SEPARATED SEWERAGE SYSTEMS

In Section 3.2, the effects of the sewer system itself on flow quantities and qualities are discussed. Such effects are studied by routing the flow hydrograph and pollutograph (i.e., variation of constituent concentrations with time) through the sewer system.

Section 3.3 makes a brief reference to the hydraulic design of sewer systems and recent developments in this field.

Finally, computer models which can be utilized for the analysis and design of urban drainage systems are discussed in Section 3.4.

3.1 Quantity and Quality of Flows Entering Urban Drainage Systems

This section examines in detail the following flows entering urban drainage systems:

- rainfall-runoff,
- sanitary sewage,
- industrial wastes, and
- infiltration/inflow.

3.1.1 Rainfall-runoff

The rainfall-runoff process on an urban watershed consists of hydrologic and hydraulic components. Rainfall losses due to groundwater infiltration and surface depression storage and the resulting routed overland flow comprise the hydrologic component and can be characterized by an inlet hydrograph. The summation of inlet hydrographs and the routing of the resultant flow through the sewer network comprise the hydraulic component.

While the hydraulic aspects of flow routing through conduit networks have been understood for some time, the hydrologic phenomena, involving much more complex interrelationships between many different physical processes, are not well understood.

A continuity equation for the precipitation-runoff process can be written as:

$$V_p = V_r + V_e \tag{3-1}$$

where: V_p = the volume of precipitation,

V_e = the volume of precipitation losses, and

V_r = the volume of runoff.

Typically, runoff is determined from a known precipitation by estimating the precipitation losses and their variation in time. Much of the hydrologic analysis then deals with the estimation of these losses.

Urban watersheds consist of two types of drainage elements - surfaces on which runoff is generated, and collecting channels, such as gutters, laterals, main and trunk sewers, which convey runoff. Surfaces are either pervious or impervious. Impervious areas may or may not be directly connected to the sewer system. Pervious areas may also contain some scattered impervious areas which are not connected to sewers. The degree of urbanization of a catchment is then defined by the catchment imperviousness, which is the ratio of the directly connected impervious area to the total catchment area.

During precipitation on a pervious surface (Figure 19), water is continuously infiltrating into the ground. Other losses, such as interception and evapotranspiration, are usually neglected in urban hydrology.

Various mathematical formulations of the infiltration process have been developed. Among these, Horton's equation is particularly popular [1]:

$$F = f_c + (f_o - f_c)\, e^{-\alpha t} \tag{3-2}$$

where: F = the infiltration capacity (in/hr);

f_c = the final infiltration rate, $t \to \infty$ (in/hr);

f_o = the initial infiltration rate, $t = 0$ (in/hr);

α = the decay coefficient (min^{-1}); and

t = the time (min) measured from the beginning of rainfall.

The actual amount of infiltration also depends on the relationship between the total precipitation and total infiltration capacity. If at a certain time the total precipitation is less than the total infiltration capacity, the actual infiltration may exceed values calculated from the above equation. The localized runoff from pervious areas starts only when precipitation exceeds infiltration capacity. Such an excess flow then starts to fill depression storage.

Depressions on natural surfaces vary greatly in geometry. While there may be some instantaneous runoff from sub-areas without surface depressions, the entire area contributes to runoff only after all the depressions have been filled.

Infiltration is assumed to be nil on impervious areas. Depression storage, although much less than on pervious areas, occurs in the same manner described above for pervious areas and cannot be disregarded (Figure 19).

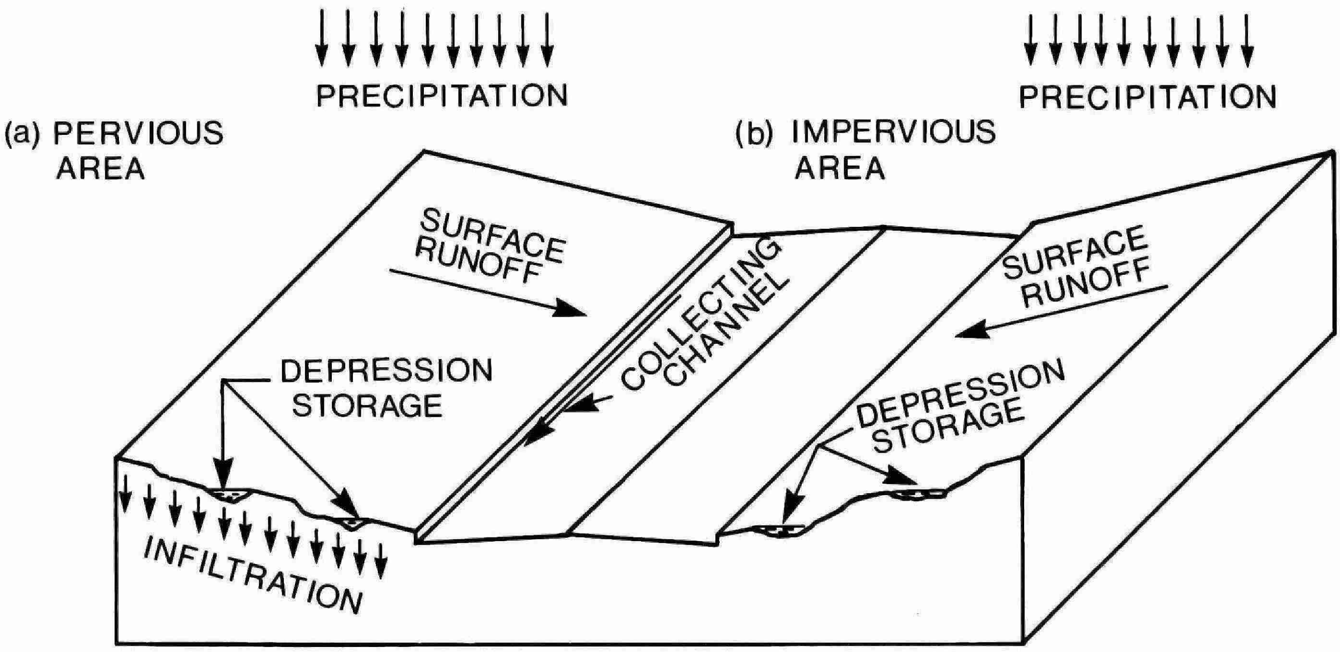

FIGURE 19. PRECIPITATION-RUNOFF PROCESS ON URBAN CATCHMENTS

As the depression storage is filled, more water becomes available to overland flow. Continuing rainfall leads to an increased depth of overland flow and increased surface runoff rates. There is a time lag between the surface element inflow (rainfall) and outflow (runoff). Such a time lag is caused by temporary storage of runoff water in the total surface storage, which consists of the depression storage and storage on the overland flow plane.

The overland flow from both pervious and impervious areas is collected in drainage ditches and gutters and conveyed to sewer inlets. Inlet hydrographs are then summed and routed through drainage networks as described in Section 3.2.

Understanding of the water quality aspects of the precipitation-runoff process is much less advanced. Rainwater reaching the surface contains various impurities in appreciable concentrations derived largely from urban air pollution. Once on the ground, the pollution load in rainwater is further augmented by dissolving and transporting substances accumulated on the catchment surface. Such processes are particularly intense on impervious areas. Eventually, an inlet pollutograph is derived and routed through the drainage system as described in Section 3.2.

Precipitation is a random event in space and time and must be described and characterized in some manner for runoff computation. Such a characterization is done in various ways, depending on the runoff computational procedure employed. For example, for the application of the Rational Method, the precipitation (or rainfall) event is described by rainfall intensity-duration curves for events of various return periods. However, in hydrologic synthesis, a storm hyetograph (i.e., variation of rainfall intensity during the entire storm) and the frequency of occurrence are required.

3.1.1.1 Selection of frequency of occurrence of the design event. In engineering calculations of runoff, it is assumed that the frequency of occurrence of a rainfall event is identical to the frequency of occurrence of the resulting runoff. Such an assumption is not correct, since a given storm may produce runoffs of various magnitudes and frequencies depending on the antecedent characteristics of the catchment. Some procedures eliminating the need to assume the identical frequencies of occurrence of the rainfall and runoff events will be discussed later.

The selection of the proper design frequency for drainage projects is a compromise between periodic inconveniences, and damages due to flooding and the cost of preventing this flooding.

Many minor drainage design projects do not warrant a detailed analysis of the cost of flood protection vs. flood damage relationship, and consequently design periods for minor drainage components are specified in municipal criteria for design of storm water drainage. In some cases, these criteria are set fairly arbitrarily.

The minor drainage system described in Chapter 2 is designed for relatively frequent storms (recurrence five to ten years). On the other

hand, the elements of the major drainage system, which are not necessarily included in the criteria, are designed for low frequency storms.

A summary of typical design storm frequencies for a number of large Canadian municipalities is presented in Table 6 [2]. The data in Table 6 represent median frequencies and most of the municipalities surveyed did not distinguish between minor and major drainage in their design criteria.

TABLE 6. TYPICAL DESIGN STORM FREQUENCIES FOR CULVERTS AND STORM SEWERS

Storm Sewers	Culverts	
Residential, commercial and industrial districts	Driveways, major and minor roads	Freeways
5 years	5 years	10 years

The above values should be considered as guidelines only. Additional discussion of design frequencies is given in Chapter 2 and standard design manuals [3,4].

3.1.1.2 Rainfall intensity - duration curves. Traditionally, the design rainfall event has been defined by rainfall intensity-duration curves which were used in conjunction with the Rational Method for calculation of runoff (see Figure 20). Curves of this type are derived from historical precipitation records by a procedure described in detail in a U.S. weather bureau manual [5]. Bruce [6] has developed reliable curves for many locations in Canada.

Rainfall intensity-duration curves offer an adequate rainfall characterization for the Rational Method only. In the Rational Method, the rainfall intensity is assumed to be constant for a selected storm duration which equals the time of concentration of the catchment under consideration. The assumption of a constant rainfall intensity is not acceptable in some other methods of runoff computation; these require complete design storm hyetographs.

3.1.1.3 Synthetic design storms with varying rainfall intensity. The hydrologic synthesis approach to runoff calculation requires, as an input, a storm hyetograph in which rainfall intensities vary with time as

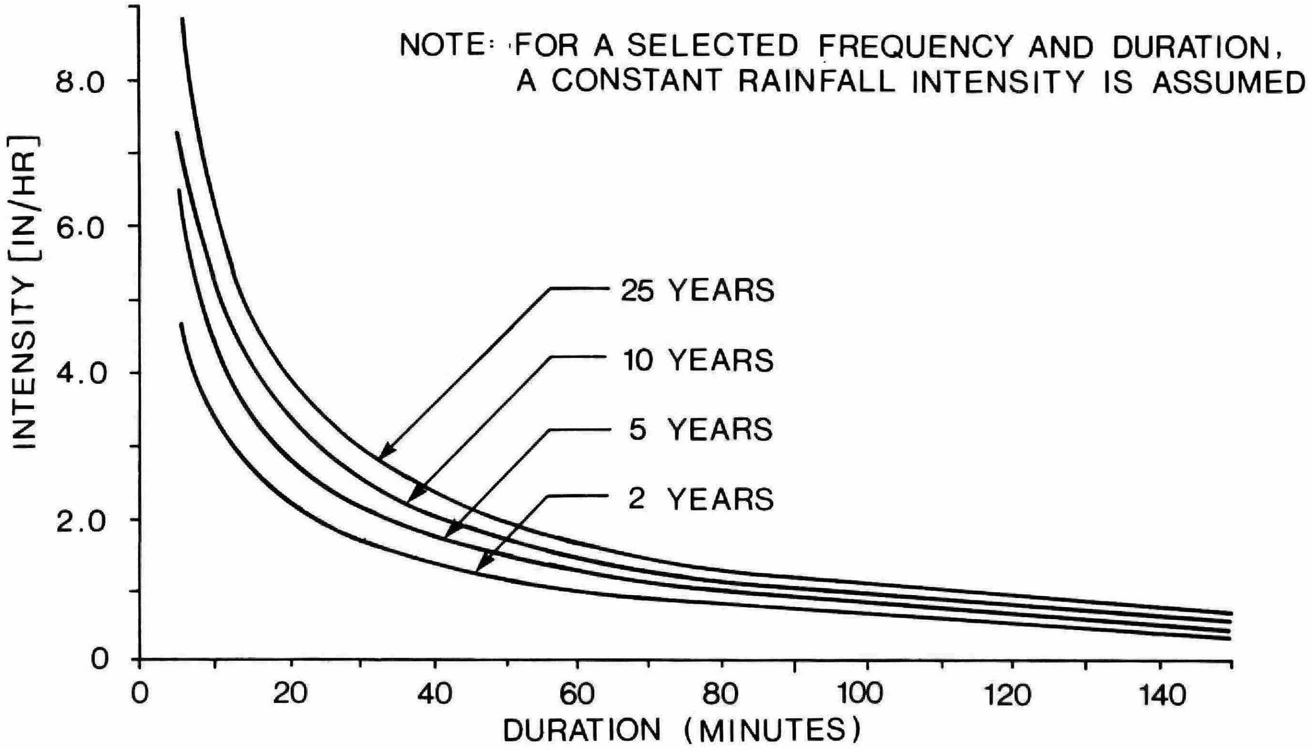

FIGURE 20. INTENSITY-DURATION RAINFALL CURVES, MALTON, ONTARIO [5]

observed in nature. Such a hyetograph may be selected from the historical precipitation record, or may be derived by synthesis and generalization of many observed events. In the latter case, a synthetic design storm of estimated frequency of occurrence is derived. The concept of synthetic design storms is subject to criticism. McPherson [7] pointed out that attempts to assign a mean frequency of probable occurrence to a design storm are meaningless because of statistical nonhomogeneity of rainfall, runoff, and quality, and the neglect of the effects of prior storms on the runoff from a given storm. At best, the resulting time and spatially variable design storm pattern represents a statistical summary of historical precipitation records. Since the theory of synthetic design storms is undergoing rapid development, future revisions of this subsection will be necessary.

In a study for the City of Hamburg [8], the following steps were taken to develop a synthetic design storm.

- derive a set of rainfall depth-duration-frequency curves;
- derive area-depth-frequency curves;

- establish a temporal rainfall intensity distribution;
- establish a spatial distribution of rainfall intensity.

Depth-duration-frequency curves may be readily derived from the intensity-duration-frequency curves, or from precipitation data using the methods described earlier.

In the next step, an area-depth relationship is found for the basin for events of various frequencies. This relationship is required only for catchments with areas larger than several km^2, otherwise point precipitation is acceptable. The precipitation depth reduces somewhat with an increasing catchment area. If sufficient local data are not available for this analysis, the U.S. Weather Bureau charts [9] may be used.

The variation of precipitation with time during the storm is determined in the next step by using some of the rainfall patterns reported in the literature, or preferably, by deriving such a pattern from the local precipitation data. Among the temporal rainfall patterns reported in the literature, the best known appear to be the Chicago design storm [10], the U.K. Meteorological Office design storm [11], and the distributions reported for the State of Illinois [12]. The general applicability of these distributions has not been studied and therefore it is preferable to derive such distribution by means of a statistical two-fold analysis of variance performed on the local data [8]. The resulting distribution is then defined as a sum of three components - a random term, a probability of exceedence term, and an average time distribution term.

Finally, the spatial rainfall intensity distribution is determined by a three-dimensional statistical analysis of the problem; two dimensions refer to space and one is an occurrence component [8], representing the recorded rainfall depth for a certain event and duration at each gauge.

The development of a design storm may be substantially simplified for small catchments (areas of the order of several km^2) because the areal distribution effects become negligible and the problem is reduced to defining the precipitation depth and its variation with time.

A comparison of the Chicago and Hamburg design storms is shown in Figure 21. A large discrepancy between the storms is obvious and illustrates the subjectivity of various approaches to the definition of design storms.

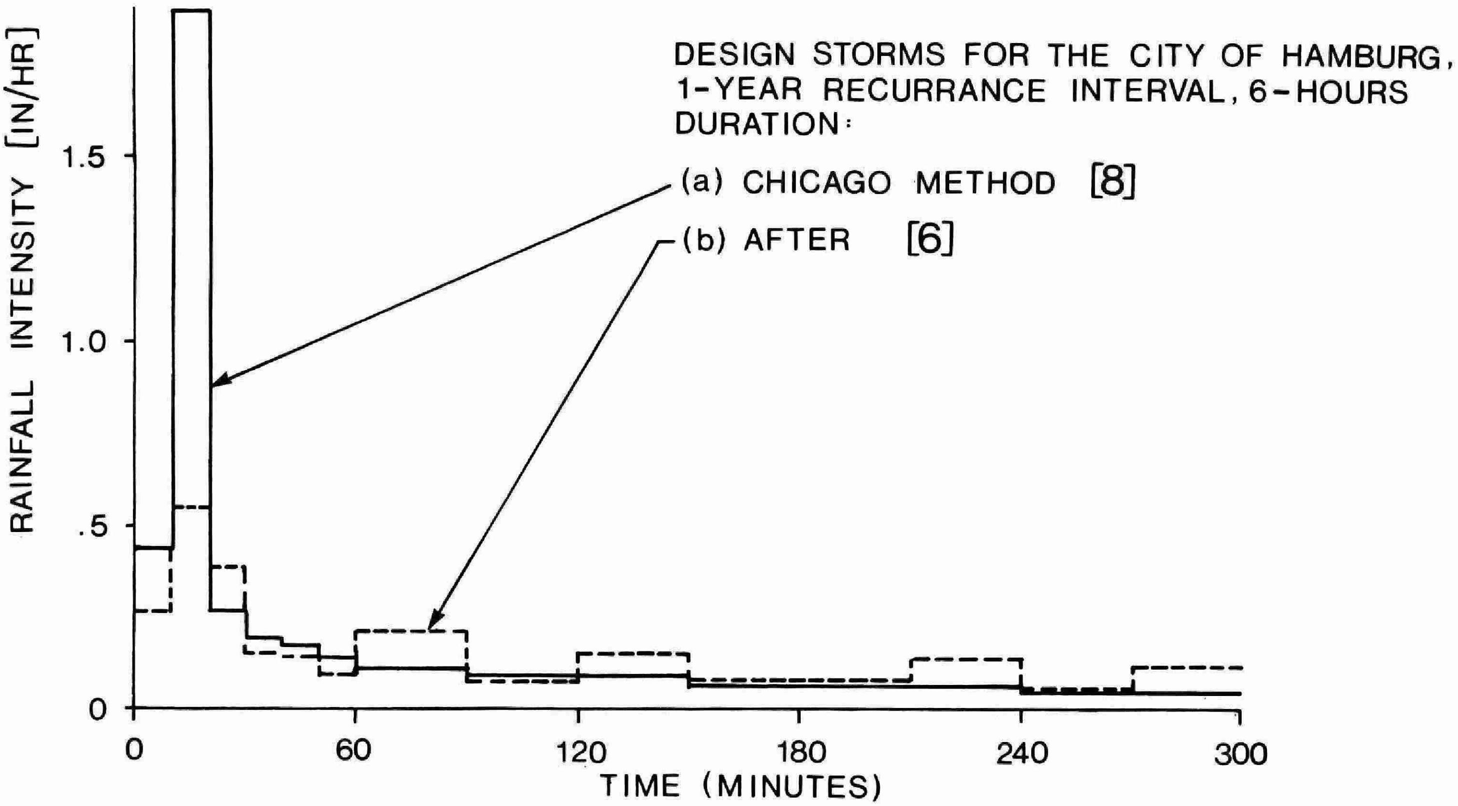

FIGURE 21. COMPARISON OF TWO DESIGN STORMS [6]

3.1.1.4 Historical design storms. Difficulties with the derivation of synthetic design storms, as well as uncertainties involved in these storms, led to the development of an alternative approach - adoption of a historical design storm. The selection of such a storm is done either directly or indirectly.

The direct method selects a historical storm for which characteristics are well-documented and for which the drainage system behavior is also known (e.g., the extent of flooding and damages). The same event may also be used as a regional storm (e.g., Hurricane Hazel). The frequency of occurrence of these historical events can be estimated. The approach described may be applicable to major drainage elements. Minor drainage elements are typically designed for more frequent events.

The indirect method is based on the frequency of occurrence of runoff events. A precipitation record is first translated by means of a simple continuous simulation model into a runoff record. A statistical analysis of the runoff record is then performed and the frequency of

occurrence of various runoff flows is determined. For a selected runoff flow the corresponding storm can be identified and used as a historical design storm on catchments of similar size in the study area.

3.1.1.5 Risk-based design. Risk-based design of large hydraulic projects is well accepted in engineering practice. A similar approach was proposed for storm sewers by Tang and Yen [13] but is not yet common in engineering practice. It is believed that a risk-based design would be particularly useful for drainage systems in which considerable flood damages can be expected.

The procedure considers uncertainties involved in runoff computations, such as the Rational Method or any other technique. A design safety factor, SF, is defined as:

$$SF = Q_c / Q_d$$

where: Q_c = the sewer pipe capacity, and

Q_d = the design flow.

Using the Rational Method and rainfall intensity-duration-frequency curves, Tang and Yen [14,15] established the risk vs. safety factor curves for a particular location. After designing the entire sewer system, risks associated with the individual pipes can be evaluated from the risk vs. safety factor curves. Should these risks be too high, the system is redesigned by either selecting a lower frequency storm, or by selecting the maximum acceptable risk and deriving the corresponding safety factor.

Note that when the maximum acceptable risk is specified, the design event frequency may vary during the calculation. The reduction of a maximum acceptable risk for a particular pipe is the same as designing the sewer pipe for a lower frequency storm and reducing the safety factor.

The risk-based design can be applied in conjunction with any arbitrary runoff computation procedure. The optimization of the risk-based design will be discussed later.

3.1.1.6 Design storm for water quality-oriented design. The preceding paragraphs on design storms dealt exclusively with runoff quantities. When water quality aspects are to be considered, a special analysis of the precipitation data may be required. The frequency of occurrence of pollution loads of a certain magnitude differs significantly from the

frequency of occurrence of the corresponding storm. The total pollution load produced by an event depends not only on the event itself, but also on the length of the antecedent dry weather period. Consequently, the design storm approach is rarely used in quality-oriented drainage design. Instead, a continuous simulation of runoff quality and the associated costs of quality control are more frequently used and these provide a good basis for selecting a cost-effective means of runoff quality control. Alternatively, single-event simulations may be performed for a series of typical storm events of return periods varying from several days to several years. The drainage system response to these events is determined and a cost-effective runoff quality control measure is selected. In other cases, the selection of the quality control measure may be based on the desired degree of protection of the receiving waters.

While the quantity-based design calls for design events with return periods of the order of several years, the most cost-effective measures for the control of runoff quality may be obtained for events with return periods of several weeks or months. Such measures are likely to control the first flush pollution even for storms with much longer periods of recurrence.

When studying the water quality aspects of urban drainage, it may also be necessary to consider the quality of rainwater. In particular, nitrogen and phosphorus concentrations in rainwater may be appreciable. Extensive data on precipitation chemistry in Canada are contained in three recent publications [16,17,18].

3.1.1.7 Urban runoff quantity computation. Quantitative estimates of runoff from a drainage area can be made by using one of the following four basic techniques, or combinations thereof:

- empirical formulas (the Rational Method is most widely accepted),
- statistical regression techniques,
- frequency analysis of stream flow, and,
- hydrologic synthesis.

Rational Method. The following description of the Rational Method was adopted from the manual, "Residential Storm Water Management" [4].

A wide range of empirical formulas relating runoff to rainfall have been developed over the years. The oldest of these is the Rational Method which, together with its derivatives, forms the basis for much of urban hydrology as currently practised. The Rational Method is presented below in some detail, since it is the most popular method and may be useful in the detailed design of minor components where a high degree of accuracy is not required. Note also that the uncertainties involved in the Rational Method can be taken into account as pointed out by Tang and Yen [13].

In the Rational Method, the peak runoff, Q, in cubic feet per second, is computed as:

$$Q = CiA \tag{3-3}$$

where: C = runoff coefficient representing the characteristics of the drainage area;

i = average intensity of rainfall, in inches per hour, for a duration equal to the time of concentration, t, for a selected rainfall frequency;

t = concentration time in minutes, defined as the time lapsed between the beginning of rainfall and the occurrence of the runoff peak at the point under consideration; and

A = size of drainage area in acres.

Guidance for selection of coefficient C is provided by Table 7 which shows commonly used values in accordance with the type of development and local soil characteristics. A composite C value should be weighted in proportion to the acreage in each part of the subdrainage area.

It has been common practice in determining the time of concentration, t, at a given design point to specify a fixed time for the flow to reach the first inlet (the so-called inlet time) and then to add the time of travel in the pipe system to that point. The time of concentration for the overland flow can be estimated from various formulas [20] or from Figure 22 [21]. In determining the time of concentration for downstream locations, paved gutter, swale or channel velocities may be estimated by making a preliminary estimate of their discharges and using open-channel

TABLE 7. RUNOFF COEFFICIENTS [3,4]

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighbourhood	0.50 to 0.70
Residential	
Single Family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemetries	0.10 to 0.25
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30
Character of Surface	
Pavement	
Asphalt or Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.70 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average 2 to 7 percent	0.10 to 0.15
Steep, 7 percent or more	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent or more	0.25 to 0.35

Remarks

a) The coefficients in this table are only applicable to storms of five to ten-year return frequencies, and were originally developed when many streets were uncurbed and drainage was conveyed in roadside swales.

b) For recurrence intervals longer than ten years, the indicated runoff coefficients should be increased, assuming that nearly all of the rainfall in excess of that expected from the ten-year recurrence interval event will become runoff.

c) The runoff coefficients indicated for different soil conditions reflect runoff behavior shortly after initial construction. With the passage of time, the runoff behavior of sandy soil areas will tend to approach that of heavy soil areas. If the designer's interest is long-term, the reduced response indicated for sandy soil areas should be disregarded.

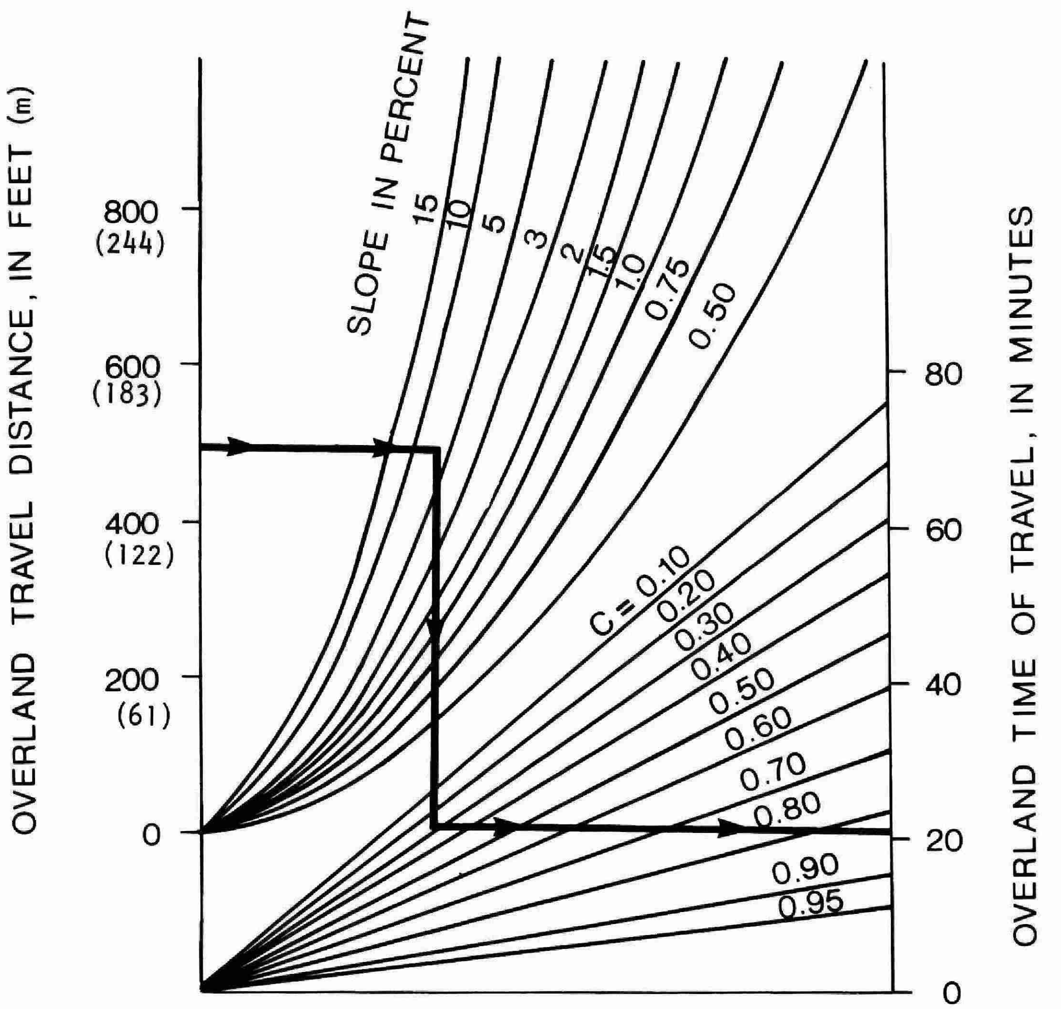

FIGURE 22. RELATION OF OVERLAND TIME OF TRAVEL TO OVERLAND TRAVEL DISTANCE, AVERAGE OVERLAND SLOPE, AND COEFFICIENT C - FOR USE IN RATIONAL METHOD [21]

flow charts published by the U.S. Bureau of Public Roads [22]. Travel time is then computed using these velocities.

Appropriate values of rainfall intensity, i, may be available from local studies or obtained from the intensity-frequency-duration data [6]. Coefficients usually used in the Rational Method must be revised if the method is used to forecast peak runoffs from very infrequent rainfall events.

For more definitive information regarding the Rational Method and its appropriate application, several works are available [3,4,15].

The Rational Method was extensively analysed and assessed by Gray [24], McPherson [23], Mitci [25], and Watkins [26]. Most of this analysis dealt with the hydrological aspects of the method and, from that point of view, the Rational Method was found to have a number of limitations. The most frequently cited limitations are:

i) The Rational Method allows one to estimate only the runoff peak flow, not the actual runoff hydrograph which is needed for design of detention and storage.

ii) The method does not account for the time and spatial variation of rainfall intensities or variation in the contributing area, and rate of contribution during a rainfall event.

iii) The lumping of physical factors affecting runoff into a single runoff coefficient C involves large uncertainties. Contrary to the assumptions in the method, the C-coefficient is not constant for various storms or, for that matter, even throughout one storm. In fact, it depends on the antecedent moisture conditions, storm pattern and the storm frequency.

iv) The estimates of concentration times by means of various empirical formulas involve large uncertainties of the order of hundreds of percent. Such uncertainties are then transferred to the average rainfall intensities calculated for these concentration times and, eventually, to the computed flow rates. Also in flow routing, flow retardation by storage and momentum of flow in channels is not taken into account.

Despite these limitations, the Rational Method is well accepted by the majority of practising engineers and is likely to remain in use. It is interesting to note that the advocates of the Rational Method do not try to defend the hydrological limitations of the method, but rather stress the advantages of the Rational Method - its simplicity, wide acceptance and "good" performance. The last attribute "good" performance, was analysed by McPherson [23] who concluded that since the Rational Method is based on hypothetical conjecture unsupported by field data, the method cannot be verified in the field. A number of factors contribute towards conservatively safe drainage design. Among these, the most important appears to be the narrow range of rainfall input. In some

cases, a $\pm 20\%$ variation in design rainfall intensities about five-year values may cover a range from two to 15-year return periods. In other words, shifting the C-values up or down by 20% (and such differences in C-values selected by various designers are common) has the effect of changing a five-year design frequency to about a 15-year or two-year frequency, respectively [23].

In summary, controversy regarding the applicability of the Rational Method is likely to continue in the near future. In the end, it is the designer's responsibility to select the best design tool for a particular project. At present, the Rational Method is typically used for a detailed design (sizing) of the components of minor drainage systems. Such a practice is acceptable provided that the limitations of the Rational Method are not exceeded. Hydrological limitations of the Rational Method become more apparent with an increasing drainage area. On larger catchments (greater than 80 hectares (200 acres) [56]) hydrological modelling is nearly always justified.

Many special design considerations, such as storage, water quality aspects, additions to the existing drainage system, and flooding in the receiving waters, require the type of information which has to be obtained by more sophisticated design tools.

Regression analysis. Regression models seek to relate a causal factor such as rainfall and/or watershed characteristics to an effect such as peak discharge, runoff volume or annual mean flows, through statistical correlation. Their applicability to urban storm water systems is minimal, mainly because of the lack of adequate historic data for regression analysis and because watershed characteristics are constantly changing when urbanization is in process. Generally, these models do not predict the total hydrograph, and may be of limited use whenever storage in the system is being considered [27].

Frequency analysis. At stations with sufficiently long streamflow records, the best means of determining the probability of peak flows for drainage design is by statistical analysis of the observed flows [27]. The paucity of urban runoff data, and shifts in the hydrologic characteristics of the watershed, and hence in the flows, due to progressing urbanization create

major obstacles to this approach. Even if a progressive change has not occurred upstream of the point of interest, a sudden shift would be caused by the urban development for which the design was developed. One can conclude that a direct determination of flow frequencies for urban drainage design is impossible. A transposition of frequency curves from a gauged developed site to an ungauged one is sometimes attempted. Such a transposition, however, may involve substantial uncertainties in flow estimates, and typically, these methods produce only peak flows for piping sizing. Such limitations may be acceptable in the planning stage of the drainage design.

Hydrologic synthesis (hydrologic modelling). The problem of flow estimation at stations with limited or no stream flow records has been of major concern to hydrologists. Many methods for hydrologic synthesis have been developed and some of these are applicable in the urban environment. Two separable tasks are typically performed in hydrologic synthesis - firstly, a rainfall-runoff relationship is established to estimate the volume of runoff, and secondly, this runoff volume is distributed in time to obtain the shape of the flow hydrograph. The techniques employed vary from a systems approach (i.e., black-box models, represented by unit hydrograph techniques) to a fully deterministic approach in which the underlying physical processes are simulated. Various combinations of both approaches are fairly frequent.

The use of unit hydrograph models in urban areas is limited by data availability. Where such data are available, synthetic unit hydrographs can be derived for given development patterns [28]. These hydrographs are quite reliable in the regions for which they were developed.

Hydrologic simulation models were discussed in the manual "Residential Storm Water Management" [4] and the following general description has been adapted from that reference. Technical details of selected models are presented in Section 3.4.

For decision-makers, designers and operators, a comprehensive mathematical computer simulation program that models quantity (flow) and quality (concentrations) during the total urban rainfall-runoff process can be an invaluable tool. Such a model can give a good representation of

the physical system. It can also be used to evaluate the physical and cost effects of alternate schemes of storm water management or pollution abatement procedures.

An urban runoff model in its elementary form is simply a group of mathematical expressions that simulate the processes of the conversion of rainfall to runoff. Models can range from crude approximations of the Rational Method to many simultaneous differential equations. Nearly all applications of detailed models require a high speed digital computer.

The physical characteristics of the tributary area and the drainage system (size, slope, land use, imperviousness, sewer characteristics) must be embodied to some degree in the input to all models. The extent of data and processing required varies with the model. Much of the data reduction is relatively straightforward (e.g., tabulation of pipe diameters, slopes, lengths of a sewer system).

Many models are designed so that if all input parameters are reasonably accurate, the physics of the processes are simulated well enough to secure satisfactory results without model calibration. Because of uncertainties in the input data and some inherent model limitations, it is normally essential that some local verification-calibration data be available for a specific application site to lend reliability to the predictions of any urban runoff model. Calibration of most models against measured rainfall amounts and associated runoff flows and flow qualities can be accomplished through adjustment of the input parameters. Quality measurements, to be of calibration value, require time-related flow measurements.

There are three categories of models: planning, design/analysis, and operation. These models have somewhat different characteristics and various models overlap on objectives to some degree.

Planning models give an overall assessment of the urban runoff problem and may also provide estimates of the effectiveness and costs of alternative storm runoff management procedures. Relatively large time steps (hours) and long-term simulation periods (months and years) generally characterize these broad-objective models. Minimum data requirements and low mathematical complexity are typical. Long-term

planning models may also generate initial conditions for input to design models. The effects of urbanization are readily computed.

Design models generally involve the simulation of selected storm events with short time steps (minutes) and short simulation periods (hours). Several such models are available and these can be used for a complete description of flow, storage, and pollution routing from the point of rainfall through the total drainage system and into the receiving waters. As with planning models, design models can be used to arrive at least-cost abatement procedures for both quantity and quality problems. Data requirements can be moderate to very extensive depending on the particular model involved.

Operational models help resolve actual control decisions during a storm event. From telemetered rain and flow gauge signals as inputs to the model, estimated system responses are projected a short time into the future. In-system storage, regulator settings, or diversions, or combinations of these, may then be employed as control options. Informational needs for operations models are much greater than for either planning or design models.

Quality aspects will become increasingly important with the growing emphasis on minimizing the ecological impact of all developments on their surroundings. In those models accommodating water quality considerations, components considered range from erosion rates and sediment loads to biochemical oxygen demand, nitrogen and phosphates.

More information on technical features of urban runoff models appears in Section 4.

3.1.1.8 Procedures for estimating storm water quality. This subsection deals with procedures used for the estimation of quality of storm water. It must be stressed that the runoff quality processes are much more complex than the runoff quantity processes and are as yet little understood. Consequently, the procedures outlined should be considered only as guidelines. Whenever feasible, field data on runoff quality should be collected to verify the computed quality and to assist in the final design of drainage facilities.

Storm water quality has been studied with various degrees of sophistication and detail, depending on the purpose of the study. Two

possible approaches - a preliminary analysis and a detailed analysis - are outlined below.

Preliminary analysis of storm water quality. When establishing an overall wastewater management plan, a wide variety of pollutant sources, including storm water and corresponding impacts on receiving waters, are examined. Pollution loads in storm water need to be estimated in a simple manner, without extensive data collection. Towards this end, the U.S. Environmental Protection Agency developed the following suitable procedures for preliminary analysis of storm water quality [29].

In a general procedure for predicting runoff quality from land use and population density [30], the loads in storm water were determined as follows:

$$M_s = \alpha(i,j) \cdot R \cdot p_1\,(PD_d) \cdot \gamma \tag{3-4}$$

$$M_c = \beta(i,j) \cdot R \cdot p_1\,(PD_d) \cdot \gamma \tag{3-5}$$

where: M_s = mass of pollutant (j) from land use (i) with separated and unsewered conveyance (lb/acre•year);

M_c = mass of pollutant (j) from land use (i) with combined sewer conveyance (lb/acre•year);

$\alpha(i,j)$ = constant for pollutant (j) and land use (i) with separated and unsewered conveyance (lb/acre•in);

$\beta(i,j)$ = constant for pollutant (j) and land use (i) with combined sewer conveyance (lb/acre•in);

R = annual precipitation (in/year);

$p_1(PD_d)$ = population function;

PD_d = population density (persons/acre), and

γ = street sweeping effectiveness factor.

Equations (3-4) and (3-5) permit an estimate of BOD_5, SS, VS, PO_4, and N loads as a function of land use, type of sewer system, population density, and street sweeping frequency. Details of the application of this procedure and numerical values of parameters in equations (3-4) and (3-5) are found in Heaney et al [30].

As an alternative, a statistical method for the assessment of storm loads was developed in a recent study [29]. In this approach, the

mean runoff loading rate, W_R (lb/day), is described by the following expression [29]:

$$W_R = 5.4 \cdot \bar{c}Q_R \tag{3-6}$$

where: $\bar{c}$ = the mean constituent concentration (mg/L), and

Q_R = the mean runoff flow (cfs).

The above expression was derived assuming that storm water flows and concentrations are independent. If data collected for specific constituents indicate they are not, the analysis may be refined [29].

The mean constituent concentration $\bar{c}$ is determined either from field data or from the data given in the literature for similar areas. Table 8 summarizes representative data from a number of sources.

TABLE 8. SUMMARY OF STORM WATER POLLUTANT CONCENTRATIONS [31]

	Storm Water Overflow Concentrations			
	Separate Drainage Areas (a)		Combined Areas (b)	
Pollutant (c)	Mean	Standard Deviation	Mean	Standard Deviation
BOD_5	27	25	108	36
COD	205	118	284	110
SS	608	616	372	275
Total Coliforms (d)	3×10^5	-	6×10^6	-
Total Nitrogen (as N)	2.3	1.4	9	6
Total Phosphorus (as P)	0.5	0.4	2.8	2.9

(a) Summary of 20 cities, storm sewers and unsewered areas,
(b) Summary of 25 cities, combined sewers areas,
(c) All units mg/L except coliforms, MPN/100 mL
(d) Geometric mean

Detailed analysis of storm water quality. Pollutant loads are introduced into storm water in three ways:

- the contamination of rainwater,
- wash-outs from the surface of urban areas (including erosion), and
- deposits in the sewer system (in catch basins and sewers).

Rainwater may contribute appreciable quantities of nitrogen and phosphorus, but negligible amounts of other parameters (e.g., BOD and solids). Some data on precipitation quality in Canada are available [16].

The most important contributor of pollutants to urban runoff is the land surface itself, primarily the streets, gutters, and other impervious areas directly connected to streets or storm sewers. Accumulation and removal of pollutants is illustrated schematically in Figure 23. Pollutants which accumulate on these surfaces come from many sources, such as construction activities, household refuse, washings from yards, animal droppings, air pollution particles and dirt, oil, and tire and exhaust gas residues contributed by motor vehicles. They are generally associated with various types of street litter, such as: rags, paper, dust and dirt,

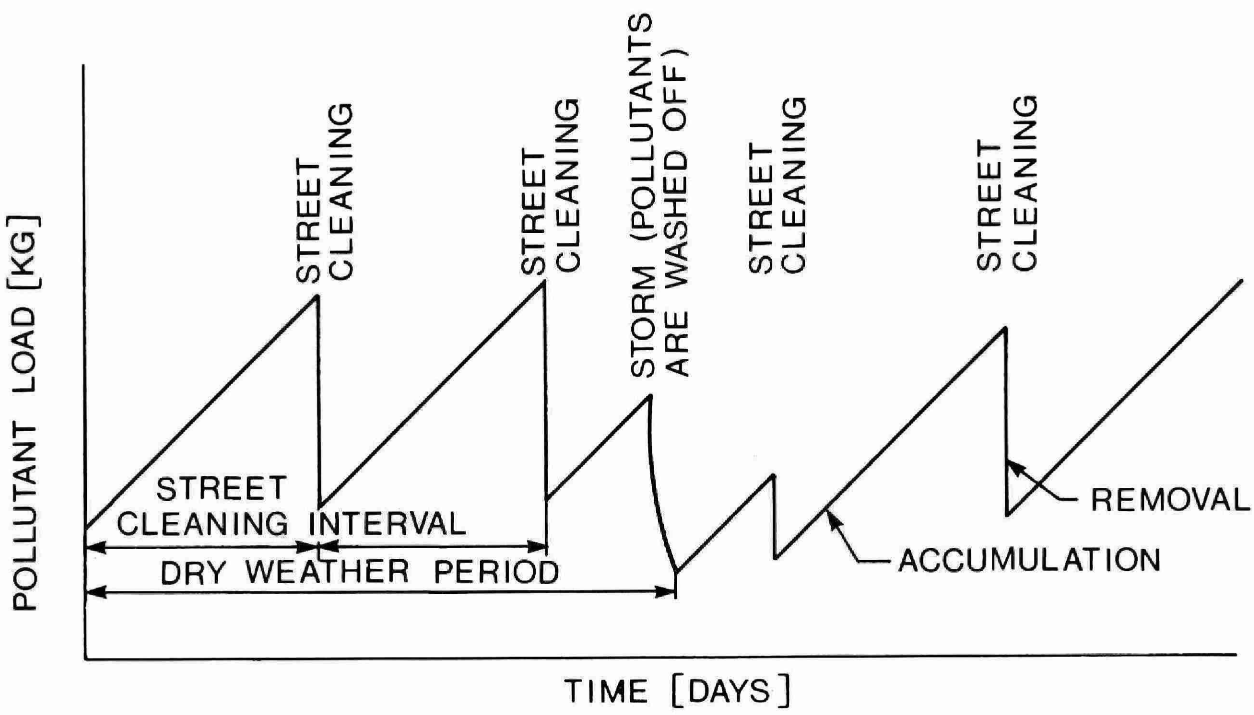

FIGURE 23. POLLUTANTS ACCUMULATION AND REMOVAL

vegetation, inorganics [32]. The most significant component is dust and dirt but in the fall, vegetation (leaves) may predominate.

Of the pollutants found in urban runoff associated with dust and dirt, COD, solids, and BOD are found in the greatest quantities, followed by nitrogen and phosphorus. Winter runoff often contains high chloride loads in areas where road salting is practised. As well, there are numerous other pollutants found in lesser quantities, such as pesticides, herbicides, residues of fertilizers and other chemicals, and heavy metals. Significant bacteria counts have also been reported for urban runoff.

There is little data on the rate at which pollutants accumulate on urban watersheds. Though the quality of combined sewer overflows (CSO) and storm water discharges has been monitored in numerous North American cities, these data are rarely usable for the calculation of accumulation rates. One of the best existing sources of information is data collected in field studies in Chicago and Tulsa [33]. A summary of these findings as well as some data from Burlington, Ontario, are presented in Table 9.

It is quite evident from several studies [34] that the rate of buildup of pollutants on an urban watershed varies significantly with land use. Industrial and commercial areas appear to be much dirtier than residential areas. This would be expected because of higher pedestrian and vehicular traffic densities in commercial and industrial areas. The data shows that pollutant accumulation rates are approximately 1.5 to 5 times as great in commercial and industrial areas as they are in residential areas [34].

Since the contribution of pervious areas to the total runoff is relatively small, the pollutant load originating on pervious areas may be neglected. In impervious areas, pollutants enter into runoff in two forms - aqueous solution, and solid particles transported by runoff.

The current approach to the estimation of pollutant buildup on urban watersheds is based on the results of the APWA study [33]. The following procedure was adopted from references describing the SWM [38] and STORM [32] models.

Since dirt appears to be the major component of street litter and the primary source of pollutants in urban runoff, one possible approach for estimating pollutant accumulation rates is to relate them to the dust

TABLE 9. RATE OF DRY WEATHER POLLUTANT BUILDUP ON URBAN WATERSHEDS [33,34,35]

AVERAGE LOAD IN kg PER DRY DAY PER km OF STREET (lb/dry day/mile)

	BOD		COD			N			PO_4		
Land Use	Chicago	Tulsa	Burlington	Chicago	Tulsa	Burlington (nitrates and nitrites)	Chicago	Tulsa (organic Kjeldahl N)	Burlington (total P)	Chicago	Tulsa (soluble orthophosphate)
Single-Family Residential*	.10 (.36)	.56 (1.98)	1.10 (3.91)	.83 (2.95)	3.92 (13.9)	.05 (.165)	.01 (.03)	.04 (.14)	.01 (.027)	.001 (.004)	.05 (.18)
Multiple-Family Residential*	.25 (.87)	.56 (1.98)	--	2.73 (9.70)	3.92 (13.9)	--	.04 (.15)	.04 (.14)	--	.003 (.012)	.05 (.18)
Commercial	.76 (2.70)	.86 (3.06)	--	3.83 (13.6)	5.72 (20.3)	--	.04 (.14)	.06 (.23)	--	.007 (.024)	.07 (.24)
Industrial	.41 (1.45)	.99 (3.51)	--	--	7.81 (27.7)	--	--	.07 (.26)	--	--	.16 (.57)

*The Tulsa data does not distinguish between single-family and multiple-family residential.

and dirt accumulation rates. The rate of buildup, DD_L, for a given land use L can be expressed as:

$$DD_L = dd_L \times (G_L/100) \times A_L \qquad (3\text{-}7)$$

where: DD_L = rate of dust and dirt accumulation on a watershed of land use L in lbs/day;

dd_L = rate of dust and dirt accumulation on watershed L in lbs/day/100 feet of gutter;

G_L = feet of gutter per acre in watershed L; and

A_L = area of watershed L in acres.

The user should supply the rate factor dd_L for his particular area. The data shown in Table 7 are default values incorporated into SWMM and STORM and may be used if no better data are available.

The initial quantity of a pollutant, p, on a watershed, at the beginning of a storm, can then be computed as:

$$P_p = (F_p \times DD_L \times N_D) + P_{po} \qquad (3\text{-}8)$$

where: P_p = total pounds of pollutant p on the watershed at the beginning of the storm;

F_p = pounds of pollutant p per pound of dust and dirt;

N_D = number of dry days since the last storm, and

P_{po} = total pounds of pollutant remaining on watershed L at the end of the last storm.

In practice, the maximum value of P_p is usually limited to the amount that would be accumulated in a 90-day dry period.

If street sweeping is practised on the watershed, the number of dry days since the last storm must be modified to account for the number of street sweepings that occurred since the last rainfall. The correct expression for P_p is then:

$$P_p = P_{po}(1-E)^n + N_S \times DD_L \times F_p [(1-E) + \ldots (1-E)^n] + DD_L \times F_p (N_D - n\, N_S) \qquad (3\text{-}9)$$

where: N_S = number of days between street sweepings;

n = number of times the street was swept since the last storm (n is an integer); and

E = efficiency of street sweeping (0.6 to 0.95).

To compute the amount of pollutant washed off the watershed during a storm, it is assumed that the amount of pollutant removed at any time, t, is proportional to the amount remaining [38]:

$$\frac{dP_p}{dt} = -KP_p \tag{3-10}$$

Since the runoff rate also affects the rate of pollutant removal, the factor K must be functionally dependent upon the runoff rate. Assuming that K is directly proportional to the runoff, R, from the impervious parts of the watershed (expressed in inches per hour) and that a uniform rainfall of 1/2 inch per hour would wash away 90 percent of the pollutant in one hour, one can say that K = 4.6R. Making this substitution into equation (3-10) and integrating over a time interval, Δt (during which R is held constant), yields:

$$P_p\,(t + \Delta t) = P_p\,(t)e^{-4.6R\Delta t} \tag{3-11}$$

Equation (3-11) is the basic form of the overland flow quality model developed by Metcalf & Eddy, Inc., as part of the U.S. EPA Storm Water Management Model [38]. Although it is simplistic and contains many assumptions it is likely the best overland flow water quality predictor or simulation model that presently exists. The pollutant washoff as described by equation (3-11) was plotted, for various values of K and constant rate of runoff R, in Figure 24.

The amount of pollutant removed during an interval Δt is $P(t) - P(t+\Delta t)$ and the rate of removal of mass from the watershed M_p is simply $[P(t) - P(t+\Delta t)]/\Delta t$, which can be expressed as:

$$M_p = P(t) \times (1-e^{-4.6R\Delta t})/\Delta t \tag{3-12}$$

Equation (3-12) must be modified, however, because not all of the pollutants tied to the dust on the watershed are available for inclusion in the runoff at a given time t.

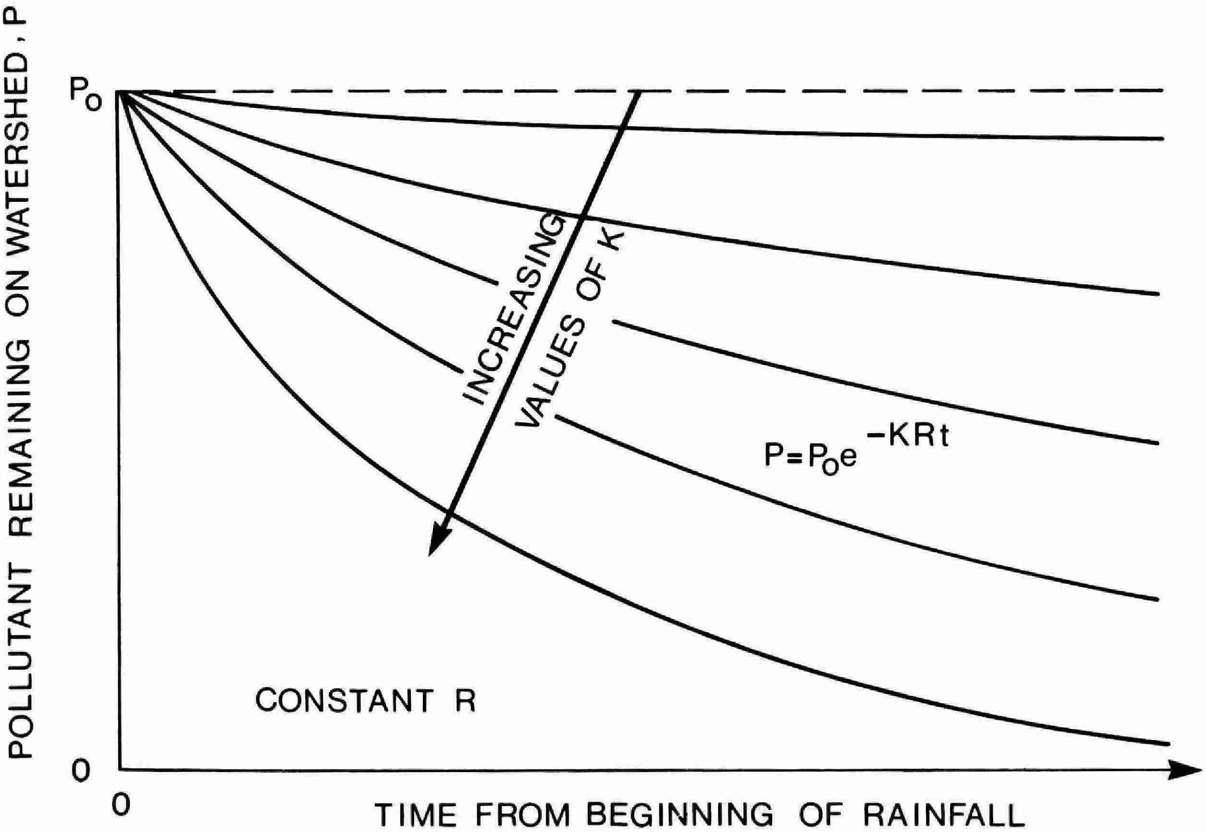

FIGURE 24. POLLUTANT WASHOFF FOR A CONSTANT RUNOFF RATE

After correcting equation (3-12) for available suspended and settleable solids and adding the BOD, N and PO_4 found in the solids, the following set of equations which are used in STORM [32] are derived.

Suspended Solids:

$$M_{sus}(t) = A_{sus} P_{sus}(t) \times EXPT \tag{3-13}$$

where: $A_{sus} = 0.057 + 1.4R^{1.1}$

$EXPT = (1-e^{-4.6R\Delta t})/\Delta t$, with Δt = 1 hour.

Settleable Solids:

$$M_{set}(t) = A_{set} P_{set}(t) \times EXPT \tag{3-14}$$

where: $A_{set} = 0.028 + R^{1.8}$

BOD:

$$M_{bod}(t) = P_{bod}(t) \times EXPT + 0.10M_{sus} + 0.02M_{set} \tag{3-15}$$

Nitrogen:

$$M_{nit}(t) = P_{nit}(t) \times EXPT + .045M_{sus} + .01M_{set} \tag{3-16}$$

PO_4:

$$M_{PO4}(t) = P_{PO4}(t) \times EXPT + .0045\,M_{sus} + .001M_{set} \tag{3-17}$$

Estimates of bacteria in storm water and combined sewage are very difficult to obtain, largely because of the highly nonconservative character of bacterial populations. The approach adopted in the SWMM is virtually identical to that used for other water quality constituents. The uncertainty involved in these calculations requires careful interpretation of the results.

The procedure outlined above is that used in the SWM model and STORM model. Other approaches have been proposed, typically based on regression analysis of existing storm water quality data [36]. Limitations of such data should be considered when data transposition to other catchments is attempted.

Urban erosion represents another important source of pollution. Quantities can be estimated from the Universal Soil Loss Equation [39]. The equation was developed as an estimate of the average annual soil erosion from rainstorms for a given upland area, expressed as the average annual soil loss per unit area, A (tons per acre):

$$A = (R)\ (K)\ (LS)\ (C)\ (P) \tag{3-18}$$

where: R = the rainfall factor;

K = the soil erodibility factor;

LS = the slope length gradient ratio;

C = the cropping management factor or cover index factor; and

P = the erosion control practice factor.

This equation represents the most comprehensive attempt to relate the major factors in soil erosion. Though sometimes used (e.g., in the SWMM) to predict the average soil loss for a given storm or time period, it is recognized that the Universal Soil Loss Equation was not developed for making predictions based on specific rainfall events. There are many random variables which tend to cancel out when computing annual time averages but which would not cancel out when predicting individual storm yields; for example, the initial soil moisture condition, or antecedent moisture condition (AMC) are parameters which cannot be determined directly and used reliably. It should be understood that equation (3-18) enables land management planners to estimate gross erosion rates for a wide range of rainfall, soil, slope, crop, and management conditions.

Equation (3-18) has found some acceptance in the United States, and apparently gives fairly good results where there are sufficient data to evaluate the constants. However, application of the equation to Canadian conditions is difficult because of variations in soil classification systems, and because of the scarcity of data for different regional conditions [24].

After the erosion rates have been calculated, they are added to suspended solids concentrations calculated for other sources. Further details are contained in a recent EPA publication [40].

The third source of pollutant loads is deposition and scouring of accumulated material in sewers and catch basins. Catch basins can be a source of first-flush pollution. An APWA (American Public Works Association) study [33] in Chicago found that the liquid trapped in catch basins between runoff events becomes septic. The liquid and solids trapped in the catch basin are then displaced by runoff water during the next runoff event. Calculation of the pollution load contribution of catch basins is described in a recent EPA publication [38].

3.1.1.9 Quality of combined sewer overflows (CSO). The quality of CSO is calculated by summing all the inflows to the sewer system and assuming complete mixing. Basically, the quality of overflows is controlled by the quality of sanitary sewage, and to a lesser degree, depending on the degree of dilution, by the quality of storm water. The resulting pollution loads are also affected by quality routing in the sewer system (see Section 3.2).

Numerous observations of quantity and quality of CSO have been reported in the literature [41]. Such data are useful for appreciation of the magnitude of pollution loads in the overflows and also for verification of computed loads. These data indicate that pollution loads in CSO typically exceed those in storm water. The overflows may also pose a public health threat because of high bacteria counts in combined sewage.

3.1.2 Sanitary sewage

Liquid and water-carried wastes from residences, commercial establishments, industrial plants, and institutions, together with minor quantities of storm, surface, and groundwaters that are not admitted intentionally, are referred to as sanitary sewage. Sanitary sewage is conveyed by either sanitary sewers or combined sewers. The sizing of sanitary sewers is based on the quantities of sanitary sewage. When analyzing existing combined sewers, both quantity and quality of sanitary sewage are of interest, particularly in the analysis of water quality aspects of CSO. Consequently, a brief discussion of quantity and quality of sanitary sewage is included here. For a detailed treatment of this subject, the reader is referred to the WPCF manual "Design and Construction of Sanitary and Storm Sewers" [3], and Ontario Ministry of the Environment "Guidelines for the Design of Sanitary Systems" [42].

3.1.2.1 Quantity of sanitary sewage. The quantity of sanitary sewage in existing sewer systems is determined by direct measurements, or similarly, as in the design of new sewers, calculated on the basis of population and population density estimates, land use, tributary area, and standard per capita sewage flows.

In the design of sanitary sewers only daily minimum, maximum, mean, and peak flows are of interest. Reported per capita sewage flows vary from 0.23 to 2.73 m^3/d (50-600 gal/day) [42]. The MOE guidelines [42] recommend calculation of the per capita average daily flow Q_{adf} by:

$$Q_{adf} = 100 \text{ gal/day/cap } (0.46 \text{ m}^3/\text{d}\cdot\text{person}) + \text{infiltration} \qquad (3\text{-}19)$$

where the recommended values for infiltration range from 0.00014 $m^3/ha\cdot s$ (0.002 cfs/acre) to 0.00028 $m^3/ha\cdot s$ (0.004 cfs/acre).

The peak flow Q_{pf}, defined as the mean rate during the maximum 15-minute flow for any 12-month period, is calculated from the following formula [42]:

$$Q_{pf} = Q_{adf} \times M \tag{3-20}$$

where the peak factor $M = 1 + \frac{14}{4 + \sqrt{P}}$ $(1.0 < M < 4.0)$,

P = population (in thousands).

In the simulation of flows in combined or sanitary sewers, the variations of sewage quantities during the day and during the week have to be recognized and quantified. If no local field data on the variation of sanitary sewage quantities are available, the variation patterns which were derived on the basis of extensive U.S. field studies [38] may be used, in conjunction with a computed or measured average daily flow, to estimate hourly sanitary sewage flows.

3.1.2.2 Quality of sanitary sewage. Quality of sanitary sewage varies widely from municipality to municipality. Where there is an existing sewage treatment plant, the sewage characteristics are best obtained from plant records or by laboratory analysis of sewage samples. In the absence of such data, estimates of quality are based on average constituent load values (weight/person/day) and sewage volumes (per person/day). The quality of sanitary sewage seems to be further affected by average family income and the use or absence of garbage grinders, and infiltration.

Typical concentrations of various constituents in municipal sewage are listed in Table 10. Constituent loads in sanitary sewage can be estimated on the basis of population and land use. Such loads are expressed in weight/person/day, and if desired, the daily average constituent concentrations can be calculated by dividing these loads by daily flows per person.

Sanitary sewage is somewhat diluted by infiltration and inflow. For the calculation of sewage dilution, it is preferable to have the sewage strength expressed in the constituent weight units rather than in concentrations. Additional details are contained in two recent EPA reports [38,40].

3.1.3 Industrial wastes

Flows from industrial areas include the process flows generated by wet industrial processes and domestic-type wastes from employees. Certain process flows are allowed by municipal ordinance to be discharged

TABLE 10. POLLUTANT CONCENTRATIONS IN MUNICIPAL SEWAGE [41]

	BOD_5 (mg/L)		COD (mg/L)		DO (mg/L)	SS (mg/L)		Total coliforms (MPN/100 mL)		Total nitrogen (mg/L as N)	Total phosphorus (mg/L as P)
	Avg	Range	Avg	Range	Avg	Avg	Range	Avg	Range	Avg	Avg
Typical untreated municipal	200	100-300	500	250-750	--	200	100-350	5×10^7	1×10^7 - 1×10^9	40	10

into public sewer systems and can affect significantly the quality and quantity of these municipal sewage flows. For modelling purposes, every attempt should be made to collect field data on the volume and composition of industrial wastes. The following factors should be considered during data collection: the volume of water, distribution in time (the periodicity of flow), the operating schedule of the industrial plant, and the pollution strength of the wastes. Additional information is found in a WPCF manual [43].

3.1.4 Infiltration and inflow

For the purpose of this manual, the following definitions of infiltration and inflow have been adopted from an APWA report [44].

"Infiltration" covers the volume of groundwater entering sewers and house connections from the soil, through defective joints, broken or cracked pipes, improperly made connections, manholes, walls, etc.

"Inflow" covers the volume of any kinds of water discharged into sewer lines from such sources as roof leaders, cellar and yard area drains, foundation drains, commercial and industrial so-called clean water discharges, drains from springs and swampy areas, etc. It does not include, and is distinguished from, "infiltration".

Infiltration and inflow are widespread problems adversely affecting most existing collection systems and treatment plants. A brief discussion of infiltration and inflow follows. For detailed information on infiltration and inflow, the reader is referred to an APWA report [44] and the WPCF Manual of Practice No. 9 [3].

3.1.4.1 Infiltration. Excessive infiltration may become a serious problem in the design, construction, operation, and maintenance of sewer systems. Infiltration was found to be an important source of large volumes of wastewater flow in sanitary and combined sewer systems regardless of the size of the drainage area, the type of sewer pipe construction, pipe and jointing materials, and the type of soil in which the sewers are laid [44]. Infiltration contributes significantly to hydraulic overloading of both collection systems and treatment plants. There are cases where infiltration and inflow exceed the dry weather flow. The MOE guidelines for the design of sanitary systems [42] specify the allowance for infiltration as 0.00014 to 0.00028 $m^3/ha \cdot s$ (0.002 - 0.004 cfs/acre).

Depending on the population density, these allowances may represent a large proportion of the total sanitary flow from the area (see Table 11).

TABLE 11. INFILTRATION ALLOWANCES AND SANITARY SEWAGE FLOWS [42]

		Sanitary Sewage flow from 1 ha (1 acre) at 0.455 m^3/person•d (100 gal/capita/day) in m^3/d (gal/day)				
		Population Density per ha (acre)				
Infiltration Rate m^3/ha•s (cfs/acre)	Total Infiltration m^3/ha•d (gal/acre/day)	25 (10)	50 (20)	75 (30)	125 (50)	250 (100)
0.00014 (0.002)	12.1 (1090)	11.4 (1000)	22.8 (2000)	34.1 (3000)	56.9 (5000)	113.8 (10 000)
0.00028 (0.004)	24.2 (2180)	11.4 (1000)	22.8 (2000)	34.1 (3000)	56.9 (5000)	113.8 (10 000)

The adverse effects of infiltration and inflow include:

- cost of treatment of infiltrated water,
- cost of construction of new relief sewers and treatment facilities to accommodate infiltration flows,
- reduced treatment plant efficiency,
- increased by-passing of treatment plants,
- higher pumping costs,
- basement flooding,
- cave-ins and structural failures of sewers and road structures resulting from soil washing into sewers, and
- increased sewer maintenance costs resulting from soil deposits in sewers.

To avoid the above costs, some degree of infiltration control must be exercised. In new systems, the control of infiltration is achieved by adequate design, appropriate sewer construction, and proper laying procedures and techniques.

Structural integrity, wastewater characteristics, resistance to infiltration and local soil or gradient conditions should be considered in pipe material selection. Combinations of these factors may make one material better suited than another, or preferable under certain conditions [41].

Good sewer joints are probably the most important single factor in the control of infiltration. A good joint must be watertight, resistant to root penetration, resistant to the effects of soil, groundwater, and sewage, long-lasting, and flexible [41]. New greatly improved jointing methods have been developed [41]. These methods include PVC and polyurethane joints, compression gaskets, and chemical-weld joints.

Sewer construction considerations include size of trenches, dewatering of trenches, pipe laying and assembly, backfilling, and post-construction performance tests. Major infiltration problems occur in house lateral connections. These connections are made after the sewer pipes have been laid and tested, and the connections are often made in a crude manner and are rarely inspected.

Correction of existing sewer infiltration can be accomplished by replacing defective components, sealing the existing openings, and building within the existing component [41,44]. The correction of infiltration involves a lengthy systematic plan which includes the following steps [41]:

1) identification of the drainage system;
2) evaluation of the extent of infiltration (by field measurements, or by examining treatment plant records);
3) survey of the sewer system and adjacent soils (smoke tests, soil conditions, groundwater conditions);
4) economic and feasibility study to determine the most cost-effective locations for infiltration control;
5) sewer cleaning (if required);
6) photographic and television inspection; and
7) restoration of the sewer system.

3.1.4.2 <u>Inflow</u>. Inflow sources generally represent deliberate connections of drains to a sewerage system. Such connections may be authorized and permitted; or they may be illicit connections made for the convenience

of property owners and for solution of on-property problems, without consideration of their effects on public sewer systems [41].

The nature of inflow water and its effect on sewer systems are very much similar to those of infiltration water. Ultimately, correction of inflow problems depends on regulatory and enforcement action by municipal officials, rather than on construction measures.

However, the effects of inflows into sewers can be greatly reduced. Many authorities advocate the discharge of roof water into street gutter areas -- or onto lot areas in the hope that it will percolate into the soil [44]. This reduces the immediate impact on the sewer system by reducing the flow volume. The use of pervious drainage swales and surface storage basins within urban areas also allows storm water to percolate into the ground [41]. Depressed manholes (those with vented covers in street areas where runoff can pond over the cover) can be repaired or the covers replaced with unvented covers [41]. Commercial and industrial water users should be encouraged to practice on-stream reclamation and reuse of so-called "clean water" (mostly cooling waters) to control this source of inflows.

Foundation drains are frequently connected to sanitary sewers and represent a major source of inflow. Additional information on foundation drains appears in Chapters 4 and 5.

3.2 Transport of Wastes in Urban Drainage Systems

Wastes entering the drainage system are transported through sewer networks to the point of disposal. During this process, flow hydrographs and pollutographs change in time and space. Such changes are calculated by routing the flow quantity and quality through the sewer network. Changes in runoff hydrographs due to routing through a sewer network are illustrated in Figure 25.

The routing of flow quantity is common in current drainage design and will be discussed in detail. The routing of flow quality is much more complex and is poorly understood. Only the underlying principles of quality routing will be outlined here.

3.2.1 Flow quantity routing

Flow routing in sewer networks is accomplished by solving hydraulic equations describing sewer flows. This is a very complicated

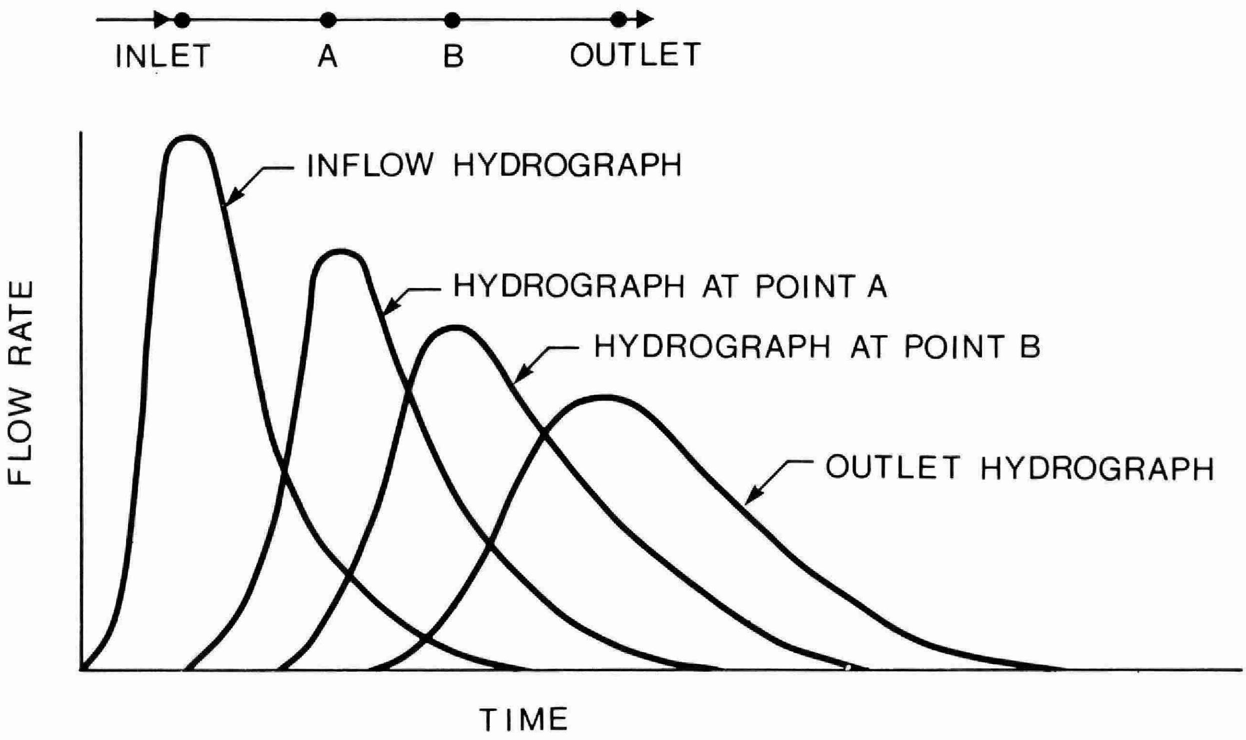

FIGURE 25. ROUTING OF HYDROGRAPHS THROUGH SEWERS

process involving the mathematical description of the drainage system, motion equations describing the flow of water, and complex numerical solutions. The procedure becomes even more complex when real sewer systems with many appurtenances, looped systems, flow reversals, sewer surcharging, and extensive backwater effects are considered. Nevertheless, all these features can be adequately described by hydraulics equations and the hydraulics of flow routing is perhaps the best understood part of the runoff process.

Unsteady, open-channel flow in partially filled sewer pipes can be described by the momentum equation:

$$\frac{\partial V_1}{\partial t} + V_1 \frac{\partial V_1}{\partial x} + g \cos\theta \frac{\partial h}{\partial x} = g (S_o - S_f) \qquad (3\text{-}21)$$

in which x is the longitudinal direction of the sewer; V_1 is the cross sectional average flow velocity along the x direction; t is the time, g is the gravitationl acceleration; h is the depth of flow measured perpendicular to x, θ is the angle between the channel bed and a horizontal plane;

$S_o = \sin\theta$ is the sewer slope; and S_f is the friction slope. The corresponding equation of continuity is:

$$\frac{\partial h}{\partial t} + D \frac{\partial V_1}{\partial x} + V_1 \frac{\partial h}{\partial x} = 0 \qquad (3\text{-}22)$$

in which $D = A/b$ is the hydraulic depth of the flow, where b is the width of the free surface.

Equations 3-21 and 3-22, known as the St. Venant equations, are first-order quasi-linear hyperbolic partial differential equations. They can be solved numerically if two initial and two boundary conditions are specified. However, difficulties are often encountered in defining the boundary conditions, and the numerical solution requires a considerable amount of computation. Hence, various simplifications are widely used to give approximate solutions. These are classified and discussed below according to their degree of sophistication.

Most of these methods can be applied to various phases of runoff, such as overland flow routing, routing through drainage ditches, gutters, pipes and storage devices.

3.2.1.1 Time lag methods. The simplest methods involve flow routing by time displacement of average inflow. These methods are generally not based on mathematical descriptions of motion or storage, but have been developed from intuitive or empirical processes.

In the simplest lag methods, the outflow hydrograph shape is identical to that of the inflow hydrograph. The lag time is calculated as the time of travel from the inlet to the point under consideration and the time of travel is typically calculated from the uniform flow equations and varies with the flow rate. To obtain a single time lag, the calculation of the time of travel is performed for a characteristic flow rate, such as the average flow for the middle 50% of the hydrograph, the flow rate corresponding to the centroid of the hydrograph, etc.

Other similar methods [45] calculate lag times for varying flows in time steps and, consequently, the shape of the hydrograph changes. Such procedures, however, complicate the computations and partly defeat the main advantage of time-lag methods - their simplicity.

The applicability of time lag methods in urban drainage design is rather limited. The method may be acceptable for runoff routing in sewer networks without significant storage and dynamic effects. The concept of time lag methods is inherent to the Rational Method runoff calculation.

3.2.1.2 Storage (hydrologic) routing. Storage flow routing is based on the flow continuity equation only; flow momentum is omitted. Storage routing is of similar complexity to the unit hydrograph methods of runoff computation.

The continuity equation is typically written as:

$$I - O = \Delta S \tag{3-23}$$

where I is the inflow, O is the outflow, and ΔS is the change in storage. To solve the equation, a relationship between outflow and storage must be specified.

Storage routing methods are sometimes divided into two categories: methods using a constant discharge-storage relationship, and methods using a variable discharge-storage relationship. Among the former, the Puls and coefficient methods are best known. The latter methods are represented by the Muskingum and Working-Value methods. Details of these and other routing methods are contained in Chow [45].

Storage routing can be applied to pipe flow as well as flow through storage devices. Storage routing is more accurate than time lag routing and may be applied to flow routing in sewer networks without extensive backwater effects or surcharging.

3.2.1.3 Hydraulic routing. In real sewer systems, various dynamic effects such as backwater and surcharging take place and affect flow through the system. These effects can be accurately described by the simultaneous solution of the momentum and continuity equations. While an accurate mathematical solution of these equations is possible for individual sewer pipes, the complexity and costs of these solutions prevent their application in general engineering practice. Consequently, numerous simplifying approximations are made in these equations as well as in their solution techniques.

Computer programs have been written for various flow routing schemes and have been built into various urban runoff models. The designer, therefore, does not need to concern himself with the mathematical formulation of flow routing and can simply select an appropriate scheme from those available. When making this selection, one should realize that the more sophisticated the routing scheme, the higher the accuracy and cost of computation. Sophisticated routing schemes are applied in the final design or analysis of major trunk sewers with extensive backwater effects, sewer surcharging, flow reversal and looping. Some of these schemes [46,47] can also simulate operation of various sewer appurtenances and operation devices, such as junctions, flow dividers, weirs, orifices, tide gates, internal storage, and pumps.

3.2.2 Flow quality routing

Quality routing is performed in conjunction with quantity routing. Experience has shown that the outflow pollutograph often differs significantly from the inflow pollutograph. As mentioned in the preceding section, runoff quality processes are very complex and not fully understood. The following discussion will therefore only summarize quality routing concepts. Because of the large uncertainty involved in runoff quality routing, this routing is quite often omitted from drainage analysis, particularly when dealing with storm sewers with medium or large slopes. On the other hand, quality routing can be fairly important in storage facilities and in combined sewers with small gradients.

Solids deposition and resuspension, and pollutant decay are important factors in quality routing in sewers.

Solids deposition and resuspension can significantly affect quality computations, particularly in the case of combined sewers with low gradients. During periods of low flow, solids settle out and accumulate in sewers. Subsequently, when flows increase above a certain threshold, scouring and resuspension of solids occurs. Solids deposition is particularly significant in storage elements (internal or external) in which the flow-through velocity, and therefore solids carrying capacity, is reduced considerably.

In solids deposition and resuspension calculations, the initial conditions at the start of the storm must be determined first. This is

done by calculating the solids accumulation since the last cleansing of sewers. During runoff events, solids deposition and resuspension can be calculated by means of sediment transport formulas. One such formula, using the Shields coefficient K as a criterion for deposition and resuspension, is built into the quality routing scheme of the SWM model [38].

The decay of nonconservative pollutants in sewers is rarely taken into consideration because of the short time of pollutant transport in sewers. For two examples of pollutant decay analysis, the reader is referred to the SWM and WRE models ([38] and [48], respectively).

Internal and external storage devices are important factors controlling sewer flow quality. Inflow velocity and therefore transport capacity are reduced substantially in these devices resulting in sedimentation of solids. Such a sedimentation process depends, among other factors, on the type of flow through the storage structure. Typically, complete-mixing or plug flow is considered, with some possibility of short-circuiting. The occurrence of a particular type of flow depends very much on the geometry of the storage device, the arrangements of inlets and outlets, the flow rate, the pollutant load, and possibly other factors as well.

The removal of suspended solids in a settling basin is a function of the overflow rate or the surface loading rate which can be expressed as:

$$V_c = \frac{Q}{A} = \text{overflow rate} \tag{3-24}$$

where Q is the rate at which the clarified water is produced and A is the surface area of the sedimentation basin. On the basis of experimental data, the following relationships were proposed [49].

$$\left.\begin{array}{l}\text{Influent suspended}\\ \text{solids concentration}\\ \text{100-200 ppm}\end{array}\right\} \text{Removal} = 0.70 - 0.4\,\frac{V_c - 300}{2\ 000} \tag{3-25}$$

$$\left.\begin{array}{l}\text{Influent suspended}\\ \text{solids concentration}\\ \text{>200 ppm}\end{array}\right\} \text{Removal} = 0.70 - 0.4\,\frac{V_c - 300}{2\ 000} + \frac{\text{SS conc x } 0.06}{190} - 0.044 \tag{3-26}$$

where V_c is the overflow rate in U.S. gal/ft^2/day. In the above expressions, resuspension of solids is disallowed and sludge removal from

the bottom of the storage device is assumed. Both expressions, which may be applied to complete mixing flow as well as to the plug flow, were used to calculate the removal efficiency vs. overflow rate curves shown in Figure 26.

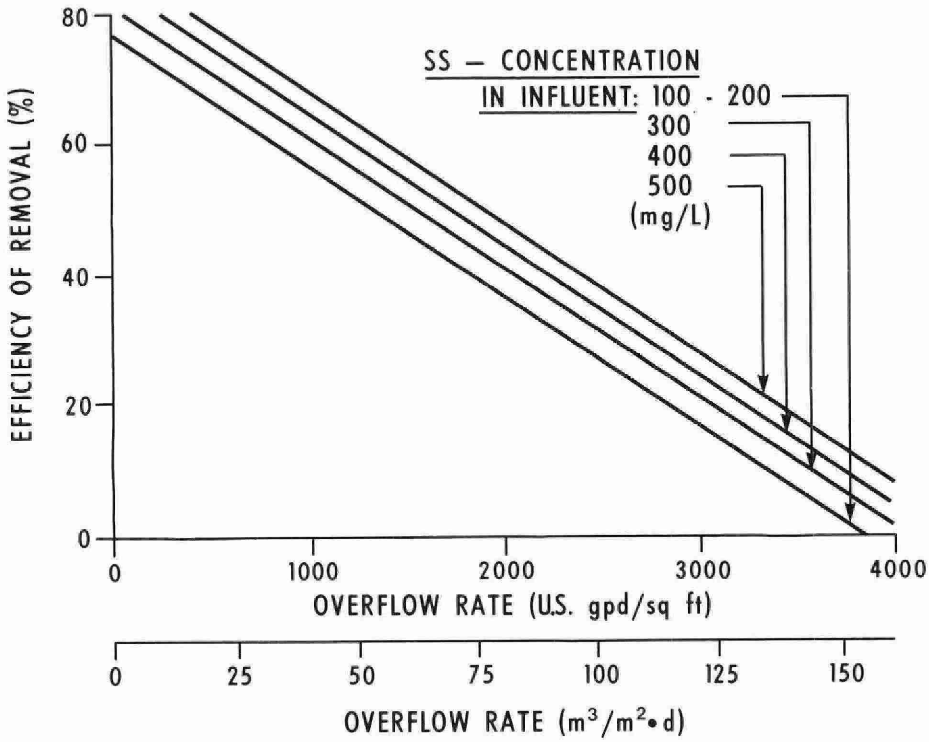

FIGURE 26. SUSPENDED SOLIDS REMOVAL EFFICIENCY BY SEDIMENTATION [40]

The time of travel of pollutants in the sewer network is calculated on the assumption that no longitudinal dispersion or diffusion take place.

In summary, water quality routing may be important in large catchments where various pollution loads arrive at the outlet at different times, in low grade sewer systems (particularly combined sewer systems) with significant solids deposition and scour, and in storage devices with appreciable detention times. Pollutant decay may be neglected in most cases, depending on the time of travel and detention in the sewer system.

3.3 Hydraulic Design of Sewers

Hydraulic design of sewers is a conventional procedure well described in various drainage manuals [3,4]. The traditional scope is

expanded here somewhat by inclusion of a discussion of the optimal design of storm sewer networks. Note that this discussion is limited to new storm sewers only and the only objective considered is flood protection [19].

Traditionally, sewer design was carried out in a piece-meal fashion starting at an upstream point and moving to the outfall. In recent years, computer programs capable of designing an optimized sewer network [19] and taking into account the effect of uncertainties on this optimization have been produced. The objective of such a procedure is to minimize the total expected sewerage system cost by selecting the optimal values of the decision variables at each stage. Techniques applicable to sanitary sewers are described in [50,51,52]. Typically, linear programming is employed.

For storm sewers, different procedures are necessary. The system cost is the sum of the cost of individual elements and each of these consists of two components -- the installation cost, which for a specified material depends on the pipe size and depth of excavation, and the expected flood damage cost. The decision variables are the drop in elevation, which specifies the pipe slope for a given length, and unless the maximum acceptable risk level is specified, the pipe diameter can also vary independently.

The optimization procedure is subject to a number of constraints because of physical limitations and acceptable practice standards. Among these constraints are the minimum and maximum flow velocities, minimum pipe size, the fact that the downstream pipe sizes should not be less than those of the preceding upstream pipes, minimum soil cover depth, etc.

This optimization problem can be solved by dynamic programming techniques. Such a solution may become very time consuming and may require a large amount of computer storage for large sewer systems. In response, an improved solution procedure, called discrete differential dynamic programming, was devised [19] and can be applied to sewer systems to produce the optimal, risk-based sewer design.

Less sophisticated design procedures are built into some existing urban runoff models [49,53].

3.4 Computer Modelling of Urban Drainage Behavior

Complete analysis of urban drainage includes consideration of a large number of parameters and factors, and such an analysis can be best handled by computer models. In some cases, all the computations described in the preceding three sections can be performed by a single computer program [38]. In the following, the capabilities and limitations of the most advanced models are described [57], followed by a more detailed description of the SWM and STORM models, and selected engineering model applications in Canada.

3.4.1 Advantages of modelling

Mathematical modelling is a developing tool that can account for differences in site characteristics and can simulate the effects of alternate development schemes, thereby obtaining qualitative and/or quantitative answers to complex planning and design problems such as the likely cause-effect relationships between urban runoff and water pollution. A large number of alternatives can be tested with relative ease and low cost over a short period of time, since model set-up costs are paid only once.

The objectives of most urban runoff studies often change with time. Mathematical models can usually be modified to meet these changing objectives, and they present an efficient means of updating plans as a result a reassessment of future conditions. The ease of modification and short response time of mathematical models make them particularly promising for use in the operational mode for real time control. Examples of such use would be computerized city-wide control of urban storm water or development of a flood warning system.

Generally, sufficient funds for immediate improvement of all parts of a system are not available. The cause-effect relationships displayed by models can be used to pinpoint those parts of the system where improvement would generate the most benefit from the funds available. A important advantage of mathematical models is that they encourage total system consciousness, rather than piecemeal design without regard for downstream effects.

3.4.2 Justification of modelling

Prior to every urban drainage study, the feasibility of using mathematical modelling should be assessed. Modelling is justified in those instances where the benefits of using a model exceed the costs of application of the model. The benefits that can accrue from the use of a model will depend heavily on the increased information that can be provided. Therefore, model calibration and verification results should be consulted prior to any practical application. In addition, during the model analysis of the collected storm water data, continual attention should be directed towards the applicability of different models to the user's problems. Finally, there are cases where urban runoff problems can be adequately solved by means of approximate empirical techniques (e.g., the Rational Method), or cases where the problems are too complex to be modelled quantitatively and, therefore, are best assessed through qualitative descriptive approaches.

3.4.3 Model limitations

Models are by necessity approximate representations of reality. It is imperative that the assumptions underlying the particular model used be kept constantly in mind; otherwise, more significance may be placed on simulation results than is warranted, or a model may be applied to a situation for which it is not designed.

Many of the current limitations of the modelling approach stem from the lack of good input and/or calibration data.

Finally, models do not make decisions; they provide qualitative and/or quantitative information to serve as input into the decision-making process. The purpose of modelling is insight, not numbers --". . . insight into how the system behaves, to what factors it is sensitive, how man may favourably or unfavourably influence those factors, and what kinds of solutions to water resource problems are likely to be technically feasible" [58].

3.4.4 Model comparisons

The following description and recommendations were adapted from Brandstetter, Fields, and Torno [54].

Mathematical models are being used more frequently for the assessment of existing sewerage system performance, the planning and

design of new facilities, and the control of untreated overflows during rainstorms. For some purposes, primarily the design of sanitary sewerage systems, steady-state models are adequate to compute the least-cost combinations of sewer pipes and slopes for specified inflows. Nonsteady-state models are required, however, to adequately analyze complex storm and combined sewerage systems under dynamic runoff conditions.

A considerable number of steady-state and nonsteady-state models have been developed in the last few years for the analysis of sewerage systems. It is consequently becoming increasingly difficult for the user to select the model best suited for a particular application. Therefore, a review of the more comprehensive nonsteady-state urban hydrologic models was conducted to develop a brief summary of the models, their features, and their strengths and limitations [54]. The models reviewed and their principal features are listed in Table 12.

In general, these models combine the runoff from several catchments and route the wastewaters within the sewer network. Most of them consider the spatial nonuniformity of rainfall; the time-varying runoff resulting from rainstorms of different intensities and durations; spatial and temporal variations in dry weather flows; the attenuation of flows during overland, gutter, and sewer conduit flow routing; and the operation of flow diversion structures and storage facilities under dynamic wastewater flow conditions. Only a few models exist, however, which also compute the water quality of the urban runoff and route the pollutants through the sewerage networks. Some models include options for dimensioning sewer pipes and two of them use mathematical optimization schemes for least-cost design of new sewerage system components. Three models have provisions for the real-time control of overflow during rainstorms. None of the models reviewed seems to have provisions for a simultaneous consideration of both the major and minor drainage systems.

A brief review of these models indicates a tremendous diversity in scope and purpose, mathematical detail, system elements and hydrologic phenomena being modelled, system size, data input requirements, and computer output. This diversity, of course, is a result of the varying conditions and objectives which govern the design and evaluation of individual sewerage systems, limitations in the available computer hardware, and progress in the state-of-the-art of modelling specific phenomena.

TABLE 12. COMPARISON OF MAJOR MODEL CATEGORIES [54]

				CATCHMENT HYDROLOGY							SEWER HYDRAULICS								WASTEWATER QUALITY									MISCELLANEOUS					
	MODEL ORIGIN	MODEL ABBREVIATION	YEAR	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	INPUT OF SEVERAL HYETOGRAPHS	SNOWMELT	RUNOFF FROM IMPERVIOUS AREAS	RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS	FLOW ROUTING IN SEWERS	UPSTR AND DOWNSTR FLOW CONTROL	SURCHARGING AND PRESSURE FLOW	DIVERSIONS	PUMPING STATION	STORAGE	PRINTS STAGE	PRINTS VELOCITIES	DRY-WEATHER QUALITY	STORMWATER QUALITY	QUALITY ROUTING	SEDIMENTATION AND SCOUR	QUALITY REACTIONS	WASTEWATER TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION	CONTINUOUS SIMULATION	CAN CHOOSE TIME INTERVAL	DESIGN COMPUTATIONS	REAL-TIME CONTROL	APPLIED TO REAL PROBLEMS	COMPUTER PROGRAM AVAILABLE
1	BATTELLE NORTHWEST	BNW	1973	●	●	●		●	●		●		●	●		●	●	●	●	●	●			●					●	●	●	●	●
2	BRITISH ROAD RESEARCH LAB	RRL	1969	●	●	●		●			●															●			●			●	●
3	CHICAGO SANITARY DISTRICT	FSP	1974	●	●	●	●	●	●	●	●					●	●									●		●	●			●	●
4	CH2M-HILL	SAM	1974	●	●	●		●	●	●	●		●	●	●		●												●				
5	CITY OF CHICAGO	CHM-RPM	1974	●	●	●		●	●	●	●		●	●				●	●	●				●		●		●	●	●		●	●
6	COLORADO STATE UNIVERSITY		1974	●	●			●			●	●					●	●															●
7	CORPS OF ENGINEERS	STORM	1974				●	●	●	●				●		●				●			●		●			●				●	●
8	DORSCH CONSULT	HVM-QQS	1975	●	●	●		●	●	●	●	●	●	●		●	●	●	●	●	●		●	●	●	●	●		●			●	
9	ENV PROTECTION AGENCY	SWMM	1974	●	●	●		●	●		●	●	●	●	●	●			●	●	●	●	●	●		●	●		●	●		●	●
10	HYDROCOMP	HSP	1974	●	●	●	●	●	●	●	●	●		●		●	●	●	●	●	●		●		●	●	●	●	●		●	●	
11	ILLINOIS STATE	ILLUDAS	1974	●	●	●		●	●		●					●	●												●	●			●
12	MIT-RESOURCE ANALYSIS	MITCAT	1972	●	●	●	●	●	●		●		●	●		●	●	●								●			●			●	
13	MINNEAPOLIS-ST. PAUL	UROM-9	1971	●	●	●		●	●		●			●															●		●	●	●
14	NORWEGIAN WATER RES.	NIVA	1974	●	●						●	●	●	●	●	●			●	●	●			●				●	●	●			
15	QUEEN'S UNIVERSITY	QUURM	1975	●				●	●		●																						
16	SEATTLE METRO	CATAD	1974	●	●	●		●	●	●	●	●	●	●	●	●	●												●		●	●	●
17	SOGREAH	CAREDAS	1974	●	●			●	●		●	●	●	●	●	●	●	●			●					●	●		●			●	
18	UNIVERSITY OF CINCINNATI	UCUR	1974	●				●	●		●															●			●				●
19	UNIVERSITY OF ILINOIS	ISS	1973	●	●		●	●	●		●	●		●	●	●	●	●											●	●			●
20	UNIVERSITY OF MASSACHUSETTS		1974	●	●			●		●	●	●					●	●										●					●
21	UNIVERSITY OF NEBRASKA	HYDRA	1974	●		●		●	●		●																		●				
22	WATERMATION	CSM	1975	●	●	●		●	●		●	●	●	●	●	●	●	●	●	●	●			●					●	●	●	●	
23	WATER RESOURCES ENGINEERS	STORMSEWER	1973	●	●	●		●	●		●	●	●	●	●	●	●	●	●	●	●		●			●	●		●			●	
24	WILSEY AND HAM	WH-1	1972	●				●	●		●																		●	●		●	

For some applications, models are available with considerably simplified mathematical detail to reduce input data requirements, computer storage requirements, and computer running time. Some models, however, include unnecessary approximations considering the present state-of-the-art of hydrologic modelling and computer capability. Other simplifications may be needed, however, for applications to real-time control of overflows which require repeated simulations within fixed-time constraints on a small process computer.

Usually the simplest model which simulates the desired phenomena with adequate mathematical accuracy should be selected. Input data requirements and computer running times generally decrease with decreasing model complexity. Some models include options to suppress portions of the simulation if only selected phenomena are of interest. Although this feature is not listed, it should be considered in the model selection. Some proprietary models have features which appear superior to the publicly available models, but a user may prefer to run his own model even if this model does not exactly meet all his requirements.

The simulation of water quality adds considerable complexity to a model, even if it routes only conservative substances. The complexity increases substantially if both storm and dry weather water quality are computed from land use characteristics. Additional complexities are added if wastewater treatment and receiving water flow and quality are modelled.

The testing and review of the models indicated that the routing of flow, although complex for looping and converging and diverging branch systems with special structures, is the best understood phenomena. The selection of particular mathematical formulations and numerical solution techniques is governed only by the preference and needs of the model developer and user. Research is required, however, to provide a better understanding of quality routing phenomena such as sedimentation and scour, and reactions and interactions between various pollutants in the sewers.

Considerable uncertainty exists in the modelling of catchment phenomena with regard to both the flow and water quality of storm and dry weather runoff. Adequate formulations for soil infiltration, the filling of depression storage, evapotranspiration, groundwater seepage, and soil moisture are extremely difficult because of the heterogeneity of catchment land uses, geometry, vegetation, and soils. The adequacy of catchment water quality computations based on land use and runoff has not been sufficiently demonstrated. Although various models have shown good agreement between measured and computed catchment runoff water quality, the comparisons have been too limited to assign confidence limits to predictions for catchments without measurements. The models are still useful, however, for understanding system behavior, the design of drainage systems, qualitative problem assessment, and the evaluation of the relative

merits of alternative wastewater management schemes. Model usefulness and accuracy is greatly enhanced by calibration against local data.

In general, there is a direct relationship between model and problem complexity and the cost of implementation and application. A model which simulates many special sewerage system facilities will be more complex in structure and require more data and computer storage than a model that computes only runoff from a single catchment without routing flows, or which routes flow only in a simple converging network without computing runoff from precipitation and land use. Efficient solution algorithms, however, may reduce this difference significantly.

Some models require only the input of typical subcatchment elements and perform hydrologic computations only for these typical subcatchments. They then consider the actual locations of all subcatchments for the overland and sewer flow routing computations. This approach can save considerable input preparation and computer running time.

3.4.5 Choosing a model

Several models stand out because of their hydrologic and hydraulic formulations, the ease of input data preparation, the efficiency of computational algorithms, the adequacy of the program output, and the availability of the program. Other models, although deficient in some of these respects, merit consideration because they contain special features which are not included in the more comprehensive models but may be required for specific applications.

The following models are recommended for routine applications [54]:

1) Battelle Urban Wastewater Management Model (see Table 12) for real-time control and/or design optimisation considering hydraulic, water quality, and cost constraints, provided the hydrologic and hydraulic model assumptions are adequate for particular applications (lumping of many small subcatchments into few large catchments, neglect of downstream flow control, backwater, flow reversal, surcharging, and pressure flow).
2) U.S. Army Corps of Engineers STORM model for preliminary planning of required storage and treatment capacity for storm runoff from

single major catchments, considering both the quantity and quality of the surface runoff and untreated overflows.

3) Dorsch Consult Hydrograph Volume Method for single-event flow analysis considering most important hydraulic phenomena (except flow reversal). A quantity-quality simulation program for continuous wastewater flow and quality analysis is now available, but has not yet been independently evaluated.

4) U.S. Environmental Protection Agency Storm Water Management Model (SWMM) for single-event wastewater flow and quality analysis provided the hydraulic limitations of the model are acceptable (neglect of downstream flow control and flow reversal, inadequate backwater, surcharging, and pressure flow formulation). A new version including dynamic wave flow routing for pressurized flow was released recently. This routing scheme was originally developed by Water Resources Engineers Inc. Their proprietary version of the SWM model is also recommended for routine applications [48].

5) Hydrocomp Simulation Program for single-event and continuous wastewater flow and quality analysis provided the hydraulic limitations of the model are acceptable (approximate backwater and downstream flow control formulation, neglect of flow reversal, surcharging, and pressure flow).

6) Massachusetts Institute of Technology Urban Watershed Model for single-event flow analysis provided the hydraulic limitations of the model (neglect of backwater, downstream flow control, flow reversal, surcharging, and pressure flow), or the use of a separate model for these phenomena, is acceptable.

7) Seattle Computer Augmented Treatment and Disposal System as an example of an operating real-time control system to reduce untreated overflows. A more comprehensive computer model simulating both wastewater flow and quality and including mathematical optimization should be considered, however, for new systems.

8) SOGREAH Looped Sewer Model for single-event wastewater flow and quality analysis considering all important hydraulic phenomena.

Other models listed in Table 12 do not provide special features which are not included in the models mentioned above. Their use may be advantageous, nevertheless, for certain applications where the model assumptions are adequate, and especially where assistance from the model developers is easily available. One such example is the ILLUDAS model, which is applicable to hydraulic design of small drainage systems. The model has a design capability and is inexpensive to use.

In general, the models listed in Table 12 provide useful tools to the engineer and planner for assessing, designing, planning and controlling storm and combined sewerage systems. It is extremely important, however, that the potential model user study the formulations of the models, and their limitations and approximations, if he is to use them in an appropriate manner. In addition, discussions with both the original developers and other model users can provide important information on new features and advice on use of the model not found in published reports. At present, models are used mainly for the planning of new urban drainage and for analysis of the existing system. Most design work is still based on traditional methods.

3.4.6 U.S. EPA Storm Water Management Model [38,40]

In the previous section, the SWM model was identified as one of the most comprehensive urban runoff models recommended for use in urban drainage studies. The model is nonproprietary, well-documented, and continuously updated under the EPA sponsorship. It has been independently studied and evaluated [47]. As well, an informal SWMM users' group meets regularly. The model consists of an executive (control) block which activates four computational blocks, Runoff, Transport, Storage/Treatment and Receiving Water. A flow chart of the SWM model is shown in Figure 27.

3.4.6.1 Runoff block. The Runoff block simulates the runoff of both water and water quality constituents from the start of rainfall on the watershed to the entry of flow into the main sewer system. For such calculations, the catchment is first divided into a number of subcatchments having more or less homogeneous hydrologic and operational (e.g., land use) characteristics. Rainfall intensity must be specified by the

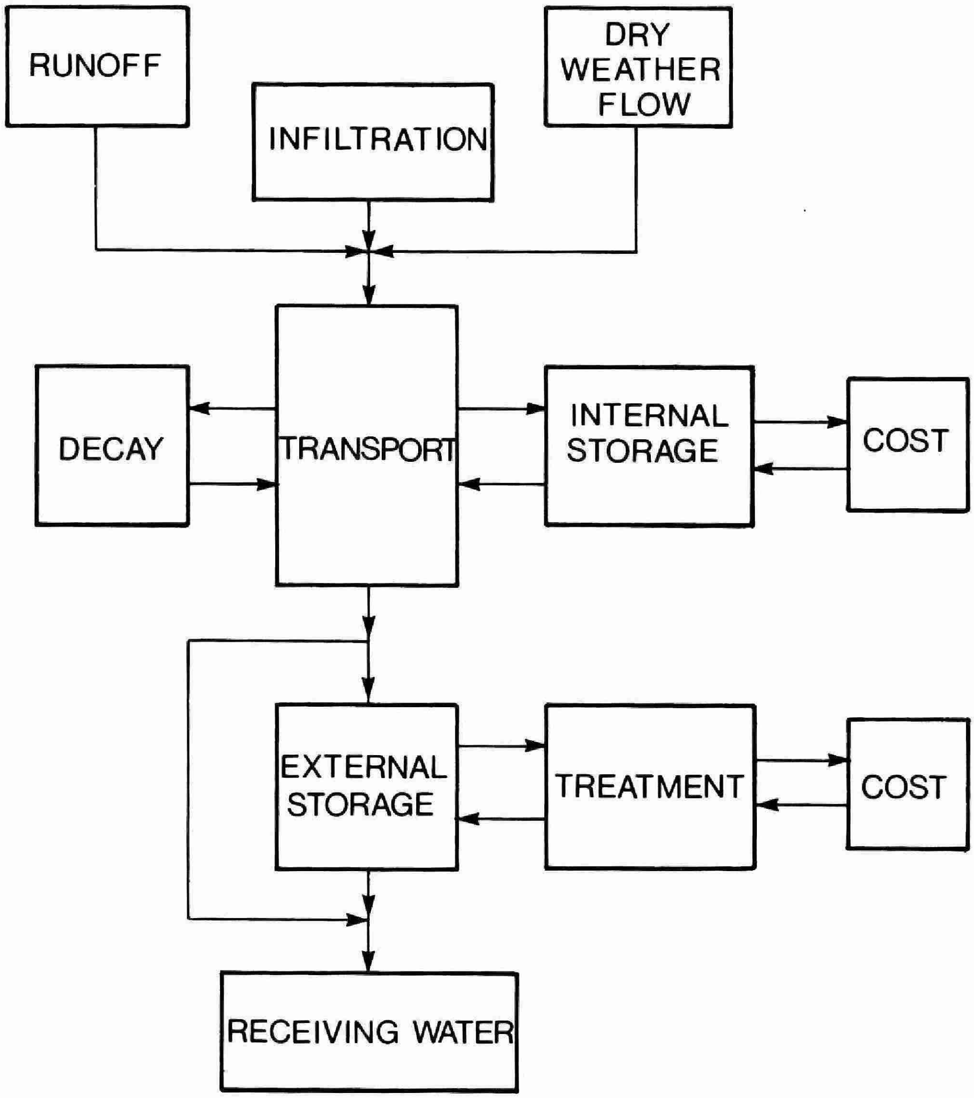

FIGURE 27. FLOW CHART OF THE STORM WATER MANAGEMENT MODEL

user, either by supplying rainfall hyetographs for individual subcatchments, or by supplying one hyetograph for the entire catchment.

Precipitation reaching the ground is reduced to allow for infiltration on pervious areas; the excess precipitation starts to fill the surface depression storage, and simultaneously, overland flow starts in areas with no surface depression storage. Overland flow is considered as gradually varied uniform flow which can be described by the kinematic wave theory.

The overland flow drains into gutters through which it is routed by storage routing techniques. Gutters in this context can be either open channels or sewer pipes (laterals).

Surface runoff quality is computed simultaneously. Pollutant accumulation on the ground prior to the storm is calculated by considering the length of the antecedent dry weather period, land use, and the frequency of street cleaning. The pollutants are assumed to be washed off at a rate controlled by the runoff intensity and time since the start of runoff. As well, specified pollution load is attributed to catch basins contents and assumed to be partly washed out during a runoff event. The contribution of soil erosion to the suspended solids load is calculated from the universal soil loss equation.

3.4.6.2 Transport block. The Transport block not only performs flow routing of sewage quantities but also performs such functions as routing of quality constituents, estimating dry weather flow, estimating infiltration (i.e., into sewer pipes), and routing flows through internal storage.

Initially, pre-storm conditions are set by computing the base flow, consisting of dry weather flow and sewer infiltration, and the associated flows of quality constituents. Dry weather flow and its quality are either specified by the user as model input, or are generated by the model. In the latter case, the data are governed by land use and any of the following land uses can be specified: single-family residential, multiple-family residential, commercial, industrial, open land, and parkland. By using general distribution functions, dry weather flow and constituent rates are specified for hours, days of week, and various locations.

Infiltration rates are calculated on the basis of existing local information on sewers, surrounding soil and groundwater conditions, precipitation, and other climatological data. Historical infiltration data can also be utilized.

Using the calculated pre-storm conditions and the runoff block input to the sewer system, routing of sewer flow quantities and qualities can proceed. The user can use either the original SWMM routing scheme, or the dynamic wave flow routine developed originally by Water Resources Engineers Inc.

The original SWMM routing scheme adopts a kinematic wave approach in which disturbances are allowed to propagate only in the downstream direction. As a consequence, backwater effects are not modelled beyond the realm of a single conduit and downstream conditions will not affect upstream computations. Surcharging is modelled by storing excess flows over and above the full pipe capacity at the upstream manhole until capacity exists to accept the stored volume. Special routing features include routing through lift stations, flow dividers, backwater elements, weirs, and internal storage. This routing scheme accurately models flows in sewer systems which do not have extensive interconnections or loops, flow reversals, significant backwater effects, and surcharging.

The second optional flow routing scheme is based on the complete dynamic wave formulation and is capable of handling such special flow conditions as looped sewer flow, flow reversal, backwater, and sewer surcharging. Usually the application of this second option is more costly.

Quality routing is also performed by the Transport block, with consideration of mixing of constituents, decay of pollutants, solids sedimentation, and scour.

The Transport block output consists of flow hydrographs and pollutographs produced at the drainage outfall or at any other desired point in the system. This output then becomes input for the Storage/ Treatment block.

3.4.6.3 Storage/treatment block. The storage submodel simulates external storage units with various inlet and outlet controls. Quantity as well as quality routing is performed and, in the latter case, plug flow and complete mixing are considered.

The treatment submodel offers seven levels of treatment which can be specified by the user. Associated capital and operational costs are also computed.

3.4.6.4 Receiving water block. Finally, the user may choose to call the Receiving Water block for an assessment of the effect of the sewage effluent produced by the preceding blocks on the receiving water, which can be either a river, lake, or estuary. Both quantity and quality simulations can be performed.

In summary the SWM model is a well-documented and proven model capable of simulating catchment hydrology, sewer hydraulics, runoff control by storage and treatment, and the fate of drainage effluents in receiving waters. The model is recommended for single-event analysis of urban drainage systems, for which the initial conditions are specified by the user.

3.4.7 Relationship between single-event and continuous simulation models

Single-event simulation using the SWMM or other detailed models provides information on the characteristics of the storm water runoff from a catchment in response to a particular rainfall event. The initial conditions must be supplied to the model. These include the number of dry days preceding the event, the infiltration potential, the available depression storage, and capacity of artificial storage facilities. Often these values must be estimated because of insufficient data, and these assumptions can significantly influence the simulated result.

The antecedent dry period controls the pollutant accumulation, while the initial infiltration and storage conditions may alter the simulated volumes and peak flows. For pollution control, statistical data describing a large number of events may be more important than a single low frequency design event. Therefore, it is often necessary to investigate the long-term runoff history and the associated pollutant accumulation and depletion. This can be accomplished using a simulation model capable of processing a long-term continuous precipitation record. From this series of events, it is possible to define a critical or "worst" overflow event or series of events in terms of total pollutant load, total volume, peak flow or as some combination of these [47].

Continuous simulation models can be used to screen a long record of runoff events to isolate critical events for subsequent single-event simulation. Information produced in this screening process may be used to set up initial conditions for detailed modelling. A methodology for the interfacing of continuous and single-event simulation models is described elsewhere [47].

A detailed review of selected continuous simulation models was given by Brandstetter [46]. The STORM model has been selected for detailed discussion here because of the ability of the model to perform both quantity and quality computations, and to simulate the effects of different storage and treatment capacities on storm water overflows. Computer time and data requirements were also considered. The STORM model is well-documented, freely available, and inexpensive to run.

3.4.8 The STORM model

The STORM model was developed jointly by the U.S. Army Corps of Engineers and Water Resources Engineers Inc. [32]. The model computes storm water runoff from a single catchment in hourly time steps based on the record of a single rain gauge. The rainfall depth in excess of the depression storage is transformed to direct runoff through the use of a specified runoff coefficient at each time step. Runoff from both pervious and impervious areas of the catchment is simulated. Snowmelt computations based upon the "degree-day" method may also be performed. The water balance between storms is determined by the recovery of depression storage based upon specified potential evapotranspiration rates.

The model does not perform routing computations, and all direct runoff computed for each time step is assumed to drain from the catchment in that time step. Various combinations of storage and treatment capacities may be modelled and the effect of these on storm water overflows investigated. Quality computations may be performed in each time step based upon the pollutant washoff equation.

3.4.8.1 Runoff quantity. In STORM, runoff is computed hourly based upon the average watershed runoff coefficient, the rainfall within the hour, and depression storage, according to the following formula:

$$R = C\ (P - f) \tag{3-27}$$

where: R = urban area runoff in inches per hour;

C = composite runoff coefficient dependent on urban land use;

P = rainfall plus snowmelt in inches per hour over the urban area; and

f = available urban depression storage in inches per hour.

The runoff generated in each hour is assumed to drain from the watershed within that hour, and may then be modified by any treatment or storage option specified.

3.4.8.2 Runoff quality. Runoff quality is also computed in hourly time steps. The rate of removal of a pollutant from the watershed within each time step is assumed to be exponentially related to the amount remaining after the preceding step. For each of five pollutants (suspended solids, settleable solids, BOD, nitrogen, PO_4) the relationship is:

$$M_p = A\ P_{(t)}\ (1 - e^{-E_u R_i \Delta t})/\Delta t \tag{3-28}$$

where: M_p = the amount of pollutant washed off in this time step, Δt;

A = the availability coefficient;

$P_{(t)}$ = the amount of pollutant on the watershed at the start of this step;

R_i = the runoff rate from impervious areas;

E_u = the urban washoff decay coefficient;

Δt = the time increment.

The user may supply the various coefficients or rely on the default values in the program. Reference should be made to the User's Manual for a more detailed description [55]. The amount of pollutant accumulation on the watershed is governed by the number of dry days, the total length of curb and gutter, the dust and dirt accumulation rate on the watershed, and cleaning practices. The various pollutants are expressed as fractions of the dust and dirt. The maximum permissible amount of pollutant is limited to that accumulated in 90 dry days.

3.4.8.3 Storage and treatment. The computation of the volumes of runoff stored, treated, and overflowed are based upon the calculated runoff and the specified storage volume and treatment rate. No storage is used until the specified treatment rate is exceeded. Subsequently, if the specified storage capacity is filled, excess runoff overflows to the receiving water body. When the runoff rate is less than the treatment rate, the excess treatment capacity is used to empty storage.

The pollutant load in the overflow is determined from a mass balance of the amount of a constituent in the runoff, the amount in storage and the amount removed by treatment. Storage and treatment are treated simply as volumetric functions.

The program provides a description of each event and a statistical summary of all events encountered in the input record. An event is considered to begin when the treatment rate is exceeded, and end when the storage unit is emptied, or when the runoff falls below the treatment rate.

3.4.8.4 Other computations and features. A new version of STORM includes dry weather flow computations and unit hydrograph techniques [55].

The shape of the watershed is not considered by STORM, nor is the time of concentration taken into account. It is assumed that all runoff flows out of the catchment during the time step in which it is generated. For larger watersheds, with a concentration time greater than one hour, the computed hydrograph will generally occur earlier than the observed one. The reverse would be true for smaller watersheds with a concentration time less than one hour. This is not usually of great concern in most studies.

Snowmelt is computed using the degree-day method, according to the formula:

$$MELT = COEF \times (T - T_T) \qquad (3\text{-}26)$$

where: MELT = snowmelt in inches over the basin;

COEF = degree-day coefficient, ranging from 0.05 to 0.15 inches per degree-day;

T = average daily air temperature, °F; and

T_T = temperature at which snow begins to melt.

Snowmelt is computed only for those days when the average daily temperature is above the temperature at which snow begins to melt, otherwise the precipitation is added to the snowpack. The computed snowmelt is distributed uniformly throughout the melting period, 9 a.m. to 5 p.m.

The Universal Soil Loss Equation [39] is used to compute soil erosion. This empirical equation was developed for agricultural land and may require calibration. However, the method is an approximative technique which may be used to define ranges or limits of watershed sediment yield.

The runoff model is calibrated by varying the following runoff parameters:

a) runoff coefficients for pervious and impervious areas,
b) evapotranspiration rates,
c) depression storage,
d) rainfall reduction factor used to relate point precipitation to basin average rainfall.

Because the model considers the entire watershed as a single computational catchment, it is quite sensitive to each of these parameters and rapid calibration for period totals is generally possible.

In summary, the STORM model is an inexpensive versatile modelling tool recommended for preliminary planning of storage and treatment capacities required to control runoff from a single major catchment. Both the quantity and quality of surface runoff and combined sewer overflows are considered.

3.4.9 Engineering applications of urban runoff models in Canada

The following examples of recent Canadian engineering applications of urban runoff models, reported in a recent survey [59], indicate the progress made in Canadian drainage design in the last several years and demonstrate the potential of urban runoff models. The projects are listed alphabetically, according to their location.

Edmonton (Alberta). The City of Edmonton engaged a consultant to carry out a master drainage study of a 4500-hectare (11 000-acre) area served by combined sewers. Two subareas were studied in detail. The WRE-SWMM was applied.

Halifax (Nova Scotia). The City of Halifax engaged a consultant to study the feasibility of storm water retention in the Kearney Lake Road area. The Illinois Urban Drainage Area Simulator (ILLUDAS) was used in this study.

Hamilton (Ontario). The Hamilton-Wentworth Regional Government engaged a consultant to study combined sewer overflows and urban runoff on two study areas. The STORM, SWMM, and WRE-SWMM were applied in this study.

Humber River Mouth Study (Toronto, Ontario). The Metro Toronto Region Conservation Authority commissioned a study of combined sewer overflows, their control, and their effects on the Humber River. SWMM was applied in this study.

Merivale Industrial Park Study (Merivale, Ontario). The Regional Municipality of Ottawa-Carleton, the Canada Mortgage and Housing Corporation and the Urban Drainage Subcommittee engaged a consultant to study the effects of a new industrial park on runoff volumes and quality, and to investigate runoff control alternatives. STORM and SWMM were used in this study.

Midland (Ontario). The Town of Midland engaged a consultant to model urban runoff and its effects on an urban lake. The STORM model was applied in this study.

Mississauga (Ontario). The City of Mississauga engaged a consultant to analyze the existing storm sewer system in the Port Credit area and to develop relief sewer alternatives. The WRE-SWMM was applied in this study.

Oil Terminals Study (London and Toronto, Ontario). The Petroleum Association for Conservation of Canadian Environment Commissioned a study of pollution loads in surface runoff from oil terminal sites. The modelling work was done with the SWMM and was supported by field measurements.

The Province of Ontario. The Urban Drainage Subcommittee engaged a consultant to evaluate the magnitude and significance of pollution loading from urban storm water runoff in Ontario. As a part of this study, runoff

simulations on four urban areas were performed with the STORM model to estimate pollution loadings and to evaluate various storage/treatment alternatives. Technologically efficient combinations of storage and treatment were identified and priced. The results were presented in a normalized form and can be used by engineers and planners to derive preliminary estimates of costs of runoff pollution control.

North Pickering (Ontario). The Ministry of Housing engaged a consultant to carry out an environmental assessment of the impact of urbanization on several watersheds in the North Pickering area. A Distributed Hydrologic Model developed by the consultant was used.

St. Catharines (Ontario). The City of St. Catharines engaged a consultant to study wet-weather flows in two sanitary sewerage areas. A proprietary model, SWAN, was used in this analysis.

Toronto (Ontario). The City of Toronto was the first Canadian municipality to use urban runoff hydrograph models extensively. The City has engaged a consultant to carry out the design/analysis of a number of drainage districts. The proprietary Dorsch HVM model has been used exclusively in these studies. The Dorsch HVM pressurized flow routing capability was the main reason for selecting this model.

Toronto Airport Study (Ontario). A study of environmental problems at the Toronto International airport was commissioned by the Department of Environment. The study included some limited monitoring of runoff quantity and quality, and runoff modelling with the STORM and SWMM.

Vancouver (British Columbia). The City of Vancouver and the Greater Vancouver Sewerage and Drainage District carried out a demonstration study with with Dorsch HVM model on a 333-ha area. Later, the results obtained with the HVM on one of the sub-catchments (38 ha, 78% impervious) were compared with those produced by the SWMM and ILLUDAS.

Vaughan (Ontario). The Town of Vaughan commissioned the development of a master drainage concept for a proposed residential development of 1820 ha in the area of Thornhill-Vaughan. The SWMM was applied in this study.

Winnipeg (Manitoba). The City of Winnipeg engaged a consultant to analyze the combined sewer system in three sewerage districts and to develop relief sewer alternatives. The WRE-SWMM was used.

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4 SOURCE CONTROLS FOR QUANTITY AND QUALITY OF URBAN DRAINAGE

4.1 Introduction

The ideas and techniques presented in this chapter relate to means for controlling the quantity and/or quality of urban surface runoff, prior to entry to a storm or combined sewer.

Quantity refers to the total volume, peak flow, and temporal distribution of runoff. Quality refers to the total mass or concentration of pollutants in runoff.

The quantity section of this chapter includes discussion of the major-minor drainage system and other topics mainly applicable to the design of storm drainage for new developments. The quality section is also most applicable to new development, but includes discussion of some controls equally applicable in already developed areas, such as improved street cleaning practices. The chapter also discusses use of natural engineering techniques in new storm water drainage, and on-site storage.

4.2 Source Controls - Quantity

4.2.1 General

Once precipitation has reached the surface within the urban area, the potential techniques for removing it are:

- overland runoff to a suitable receiver or flood control channel,
- percolation or infiltration into the ground,
- removal by the storm water drainage system,
- removal by the sanitary sewer system,
- evapotranspiration,
- physical removal in containers (snow removal only).

The first three techniques - overland flow, percolation or infiltration, and removal by the storm water drainage system - are of practical interest for handling runoff.

In combined sewers, the mixing and conveyance of sanitary sewage and surface runoff occur by design. However, where the sanitary sewer and storm water system are separated by design, intrusion of runoff into the sanitary wastes is undesirable.

Evapotranspiration and physical removal have only limited effect or application.

The "storm water drainage system" includes the surface features of natural drainage retained after development and the man-made, largely underground, features which have been added or substituted. The particular degree of runoff control needed in new developments is best determined during the urban planning process rather than as a piecemeal activity.

The overall objective of storm water management within an urban area is to minimize storm-induced damage to public and private property. The means of achieving this objective are well understood in flood-plain management. However, the flooding damage often experienced within urban areas far removed for natural watercourses and the means of prevention are much less well understood.

This latter type of flooding is often the result of the lack of recognition, at the design stage, of the two distinct components of storm drainage systems in all urban areas. Many designers tend to think of the piped storm system as the primary drainage facility. In actual fact, such a system represents only a small portion of the required drainage capacity which must be in place to avoid serious property damage during storm events.

The purpose of the piped (minor) system of drainage is to minimize inconvenience or disruption of activity from less intense storms. The major storm drainage system is the route followed by runoff waters when the piped system is either inoperative or of inadequate capacity under a given set of conditions. Detailed and correct design of the major system is becoming more and more important with the current trend towards cost reduction by using lower-intensity design storms for the minor system.

The major system is rarely deliberately designed and, as a result, storm flows beyond the capacity of the minor system take whatever route is available until a natural drainage course is reached. Typically, the excess flow will follow the road network until a low section or cul-de-sac is reached, and then will traverse private property causing damage through erosion, or by direct surface entry to structures.

Once the existence of the major and minor drainage systems is recognized, it follows that it is not economically feasible to enlarge the

minor system to obviate the need for the major system. It also follows that the major and minor systems must be properly designed to handle the flows for which each is best suited.

When redevelopment to higher population density takes place, the impact on both the major and minor systems must be considered. If this is not done, serious flooding problems may occur at a later date during an infrequent storm event.

Table 13 contains suggested criteria for each drainage system in new developments [1]. The return frequency of the design storm for the minor system could be decreased at some extra cost, for some extra convenience, without sacrificing the principle that both major and minor systems should be designed.

TABLE 13. CRITERIA FOR MAJOR AND MINOR STORM DRAINAGE SYSTEMS [1]

CRITERIA FOR MAJOR STORM DRAINAGE SYSTEM

(a) Level of Protection - from 25 to 100-year storm frequency.
(b) Continuous road grades and overflow easements to open watercourses.
(c) No flooding damage to private structures.

CRITERIA FOR MINOR STORM DRAINAGE SYSTEM

(a) Level of Service - one or two-year design curve for normal residential areas, increasing to five years for commercial districts.
(b) Design should permit surcharging to road surfaces, permitting the hydraulic gradient to follow road grades.
(c) No connections other than catch basins and inlet structures.
(d) Foundation drains may only be connected to the minor system by gravity, in such a manner that storm sewer surcharge cannot cause hydrostatic pressure on foundations.
(e) Downspouts should discharge to the ground, utilizing precast concrete splash pads.

4.2.2 The major drainage system

In new developments, the street network can be laid out to follow the natural contours of the land as a key element in design of the major storm drainage system. The roadways will then act as channels when the capacity of the minor system is exceeded. Water spread across streets must be restrained within acceptable limits and roadway "sags" cannot be permitted unless provision can be made for safe overland flow at the point of "sag". Design of drainage at street intersections requires particular care. Design and construction of reverse driveways and similar features must be carefully controlled.

Although overland flow across streets and paved areas will occur infrequently with proper design, overland runoff can become polluted. Means of minimizing the pollutants discharged should be considered and incorporated into design as necessary. Several of the techniques discussed in Section 4.3 are applicable for this purpose. The use of open channels such as street swales as part of the major drainage system offers definite advantages and opportunities for the infiltration of storm water into the ground and the interception of suspended solids and pollutants.

Additionally, "suitably designed street swales can provide up to 19 000 m^3 of runoff storage per km^2 (40 acre-ft/mi^2) contributing significantly to runoff attenuation" [2]. Open-channel drainage is particularly advantageous in situations of low density development with wide lot frontages. Open channel systems require regular public and private maintenance if they are to function as intended. "The use of enclosed components (of the storm drainage system) should be minimized to the extent consistent with the degree to which the local public will accept and act responsibly towards open channels" [2].

In many older urban areas, progressive intensification of land use, together with the lack of a planned major drainage system, has drastically reduced the local level of protection against basement flooding. In these areas, it is usually not possible to make extensive use of overland travel to open watercourses because the street layouts and grades were not designed for that purpose. Nevertheless, the possibilities of providing safe paths for overland flow should be considered when planning remedial works. In other cases, it may be possible to adopt

complementary measures to restore a reasonable level of protection in the minor system. Such measures can include increases in the degree of temporary road ponding by inlet throttling, or selective underground detention storage of road runoff.

4.2.3 The minor drainage system

A recent report produced by the Ontario Ministry of Housing [3] states that:

> "Storm drainage system costs are the largest and most variable element of residential development servicing cost now experienced.
>
> "Storm drainage practice, as usually now applied, results in systems which eliminate surface ponding, and provide for the rapid runoff of surface flows and their removal into below grade conduits under relatively short term storm intensity conditions. In providing storm service connections, the systems also enclose roof drainage, and provide foundation drain outlets to provide a degree of protection against basement seepage damage. The systems, however, usually exclude provision for the consequences of long term intensity conditions, and by their emphasis on rapid runoff and removal, exclude consideration of local and downstream ecological and environmental considerations, i.e., effluent water quality and water table relationships."

The cost of the minor storm water drainage system is necessarily increased when roof and foundation drainage must be conveyed. More seriously, in the attempt to minimize inconvenience during frequent storm events, the minor drainage system itself may contribute to structural damage to basements in less frequent, more severe events. The likelihood of significant damage is greatly lowered if the criteria in Table 11 are followed. In new developments in which the major and minor drainage systems have been properly designed there is no direct relationship between the return frequency of the design storm for minor drainage and any "expected" frequency of basement flooding - the minor system should ideally provide the same level of protection as the major.

Many variations in sewer service connections are found in existing developments for the disposal of roof runoff and weeping tile drainage. The more common variations are summarized in Figure 28 which also indicates the possibilities of property damage for each variation.

	Type of Input				Likeliest Source of Property Damage and Type of Basement Damage During Major Storm	
		Storm Flow				
	Sanitary Flow	Roofs	Weeping Tile	Roads	Structural By	Flooding By
Completely Separated	●	S	P	0	–	P*
	●	S	○	0	–	–
	●	S	0	0	0	0 **
	●	0	0	0	0	0 **
Partly Separated	●	S	●	0	–	●
	●	●	●	0	–	●
Fully Combined	⬤	⬤	⬤	⬤	–	⬤
	⬤	S	⬤	⬤	–	⬤

Key :
- ● Sanitary Sewer
- ⬤ Combined Sewer
- 0 Storm Sewer
- P Sump Pump – To Surface
- ○ Foundation Drain Collector
- S Ground Surface

* By sump overflow following Sump Pump Failure

** following structural failure

FIGURE 28. COLLECTOR SYSTEM VARIATIONS (HOUSE DRAINAGE CONNECTIONS)

The various alternatives available for weeping tile disposal are discussed in the next section. Disposal of roof leader flows to the minor system is generally undesirable in new development.

4.2.4 Foundation drainage

Foundation drains (weeping tiles) are used when it is necessary to reduce the groundwater table around basement walls to the elevation of the footings. This reduces dampness in basements and prevents structural failure of basement floors caused by hydrostatic pressure from standing water outside the walls.

Foundation drains are unnecessary when:

- the structure has no basement,
- the water table will always be below the foundation footings,
- the basement floor is designed and reinforced for hydrostatic pressure.

Most new housing in Ontario is built with basements whose floors are not reinforced, and water table conditions are such that weeping tiles are required.

Foundation drains have been connected to:

- sanitary sewers,
- sumps and sump pumps,
- separate foundation drain collector sewers,
- storm sewers.

Connection to a sanitary sewer. Connection of foundation drains to the sanitary sewer introduces an extraneous source flow whose annual magnitude and peak flow rate cannot be readily predicted. The annual magnitude and peak sustained flow rate are both of concern for pollution control. The short-term peak, lasting an hour or less, may be a significant cause of basement flooding.

A study conducted in Ontario on weeping tile flows from single family houses found that weeping tiles contributed 6-10% of the annual sanitary sewage flow [4]. This study identified periods of snow melt accompanied by rainfall as producing the most prolonged weeping tile flow peak. A follow-up study [5] developed a theoretical model of the

variation of weeping tile flows and a methodology for predicting flow quantities in the absence of any special lot grading to divert surface runoff or roof leaders away from the zone of influence of the weeping tile. The procedure demands a thorough knowledge of area physiography and soil conditions, and a detailed knowledge of the way in which the weeping tile was laid.

A study of several large areas in Winnipeg found that weeping tile flows accounted for 4% of the total dry weather flow [6]. This study also determined that, under certain conditions, peaks of 12 times the dry weather flow could occur in areas of 40 ha (100 acres) during a storm with a one-year return period. The sewage biochemical oxygen demand (BOD_5) and suspended solids (SS) peak values were above average at the maximum flows recorded, presumably because of scouring and flushing of accumulated solids. This study also noted that in areas with careful lot grading, weeping tile flow was greatly reduced even with adverse soil conditions.

In Ontario, and elsewhere in Canada, many older housing developments were designed with weeping tile flow directed to the sanitary sewer. In situations where basement flooding occurred, a common practice was to install emergency overflow structures to watercourses, many of which remain to this day. It seems logical that emergency relief should be followed by positive action to remove the problem at source, or to contain and/or treat overflows. Containment and treatment facilities for this type of overflow have been constructed in at least one instance, at Dartmouth, N.S. [7].

No comprehensive studies have been carried out to show what range of flow can be anticipated from weeping tiles if best practice, including positive lot grading, is used in design and construction. In the absence of convincing data to the contrary, it appears undesirable that weeping tile flow be directed to the sanitary sewer. Flow monitoring studies are currently being conducted on weeping tile flow from three new residential areas in Brampton, Ontario. Each area has different soil characteristics. The developments were designed using site and lot grading controls [8].

<u>Sump pumps</u>. Sump pumps are suitable for foundation drainage in many ways, but have the following drawbacks:

- they are subject to mechanical or electrical failure;
- they may be 'short-circuited' and connected to the sanitary drain by the householder, thus creating new hazards or problems;
- potential house purchasers often avoid houses so equipped, equating sump pumps with substandard servicing.

The need for house service connections for storm water can be eliminated if both roof leaders and sump pumps discharge to the ground surface. Alternatively, a shallow storm sewer service for gravity or pressure flow may be provided at the house to receive sump pump discharge. This in turn makes possible the use of shallow street storm sewers.

Storm sewers may be designed to permit surcharge to the road surface when sump pumps are used, because the sump pump eliminates hydrostatic pressure buildup on house foundations. Provision of a check valve will prevent backflow.

Connections to storm sewers. Weeping tile discharge to a storm sewer can provide a high degree of property protection when basement floor elevations are such that storm sewer surcharge to the road surface will not cause hydrostatic pressure on building foundations.

Where road surface elevation is above that of the basement floor, connection by gravity can result in substantial property damage, should the sewer surcharge above a critical elevation. While the designer can provide high 'theoretical' levels of protection, at extra cost, by designing deep or large storm sewers, there is no assurance that the level of protection can be maintained throughout the life of the development. Changes in downstream or backwater conditions, blockage or obstruction of the storm sewer, and unforeseen magnitudes of flow are some conditions which may promote dangerous surcharging. The resulting damage may range from minor cracking of basement floors and seepage, to major structural failures and basement inundation. Other approaches can offer much better property protection at equal or lower cost.

Separate foundation drain collector. A separate foundation drain collector (FDC) system is sometimes a satisfactory solution. This system uses a small third sewer laid at the same depth as the sanitary sewer for the collection and disposal of flows from weeping tiles. The extra costs of

the third sewer are offset by the fact that the street storm sewer can be placed at shallow depth, which may result in some overall cost savings.

The FDC system can provide a high level of 'theoretical' protection, given a point of discharge which reduces the risk of surcharge to negligible proportions. Because the third pipe has relatively low capacity, care must be taken to ensure that storm flows are not inadvertently misdirected into the FDC system.

The FDC system, while relatively new, has been installed in subdivisions in a number of Ontario municipalities, usually as a complete system with its own manholes [8].

4.2.5 Runoff attenuation without storage

Imperviousness, and hence the amount of runoff, increases as land use intensifies. This is generally true whether the urban development is residential, commercial or industrial. Table 14 shows typical relative magnitudes of impervious areas for three types of residential development. It also indicates that roofs and roads contribute a significant proportion of the total impervious area.

This section discusses some options available for infiltrating runoff into the ground before it enters a collection system. The measures discussed are most applicable in new developments which leave a substantial unpaved pervious area - such as single-family residential subdivisions. Combinations of simple measures outlined in this section can reduce the runoff coefficient in these areas to 0.20 from the usual 0.45.

In commercial, industrial, and other highly paved areas careful design can decrease the total runoff by infiltrating water from some of the areas usually thought of as totally impervious - driveways and parking lots for example.

Proper implementation of a comprehensive site and lot grading plan in residential and other areas can greatly assist in ensuring that infiltration is maximized by the use of features such as shallow swales for rear lot drainage, extended overland flow paths, etc.

As shown in Table 12, roofs may contribute 50% of the impervious area in urban developments. Inevitably, drainage costs are increased in areas where roof runoff is conveyed swiftly and directly to the storm drainage system. As well, the opportunity may be lost for recharge of

TABLE 14. DISTRIBUTION OF PERVIOUS AND IMPERVIOUS AREAS IN RESIDENTIAL DEVELOPMENTS

Catchment Description	Total Area Hectares	(Acres)	Impervious Area %	Percentage Distribution of Impervious Areas			
				Roofs	Roads	Sidewalks	Driveways
Low-density Single-family	23.3	(57.6)	34.0	41.6	34.1	8.5	15.80
Medium-density Single-family and Semi-Detached	19.5	(48.2)	48.3	50		30	20.0
Townhouses 6.5 per ha (16 per acre) not stacked	2.7	(6.7)	47	47	15	2	36

substantial amounts of groundwater at virtually no cost. The solution, which should be adopted whenever possible, is to direct roof leaders onto splash pads on the ground around the house, paying due attention to site and individual lot grading to avoid local ponding and seepage into basements.

Where roof leaders are connected by weeping tiles to the storm sewer, the risk of sewer surcharge in an extreme storm is increased. In hilly terrain, surcharge can result in a standing level of water outside the basement with a potential hydrostatic head of 7-10 m (25-30 ft) for a two-storey house. The result can be mild seepage, flooding, and/or structural damage. Connection of the roof leaders to the storm drainage system through the weeping tile also implies the storm sewer is deep, eliminating many lower cost servicing options.

When the roof runoff is conducted to sanitary sewers, the result can be severe overloading of this sewer system and the potential for sewage backup into basements. The large scale connection of roof leaders to the sanitary system effectively turns a separated sewer system into a combined or partly combined one, and will likely result in basement

flooding during severe storm events, overflows, and other problems in sewage transmission and treatment.

Roadways usually contribute a significant percentage of the total impervious area, as indicated in Table 14 and have also been identified as a major source of contaminated runoff [9,10].

The use of open, shallow, grassed street swales and ditches wherever possible, instead of curbs, gutters and storm sewers, can aid in reducing the total volume of runoff, slowing time of concentration, and capturing particulates. One practical limitation of the use of swales and ditches is that whenever a high percentage of road culvert must be laid, ditches may become both unsightly and hard to maintain, e.g., in areas with narrow lot frontage and/or double driveways.

The imperviousness of driveways can be greatly reduced by using one or more of the following surfacing options:

- porous asphalt pavement,
- precast interlocking blocks,
- rolled brick or cinder chips,
- concrete wheel track ribbons with a cross fall to a grassed median.

A french drain is one device that can be used where the natural ground surface is relatively impervious. Trenches can be dug and backfilled with porous material to conduct surface water to a lower more porous level. If the native porous medium is at a considerable depth below an impervious surface, shafts can be dug and backfilled with a porous material.

4.2.6 On-site detention and retention

The objective of on-site storage of runoff is either to prevent storm runoff from reaching the drainage system or to change the timing of the runoff by controlling the release rate.

On-site retention is the term for total containment of runoff with no overflow from the storage basin. Water entering retention basins eventually either recharges the groundwater or evaporates.

The concept of groundwater recharge by runoff is attractive. However, the availability of suitable sites and the potential environmental

problems can limit the application of this technique in urbanized areas where runoff is often polluted with sediments, dissolved chemicals, and pesticides, which may eventually contaminate the groundwater. The potential adverse effects should be considered wherever protection of groundwater quality is an important requirement.

Retention basins are usually shallow ponds located within natural or excavated low-lying areas. Typically, they are allowed to fill during storms and drain completely between events. In some cases, shallow basins can be grassed and used for other purposes when dry. Maintaining acceptable percolation rates can involve limiting the intake of fine suspended matter, providing trenches or vertical drains filled with granular material, and periodically scarifying the surface. The latter requirement may cause an aesthetic problem necessitating landscaping and screening by vegetation to hide the basin.

Where groundwater pollution is not a problem, shallow retention facilities for disposal of storm water may be designed for aquifer recharge as a secondary benefit. Deeper facilities, occuping less land but requiring more maintenance can also be constructed in aquifer recharge areas. Over 2 000 single-purpose recharge basins have been built in Long Island, N.Y. since the 1930's and a detailed design method for this type of basin has been published [11].

On-site detention describes the technique of delaying or retarding the rate of runoff to the collector system by containment in temporary storage.

The basic requirements for use of this technique include a containment location of defined area and volume, and a suitable device to control the rate at which runoff is released. In most cases, an emergency overflow device is also needed, to ensure safe operation.

These basic requirements can be met in many locations. The advantage of street swales have already been described. Other possibilities include:

Roof Storage - Most industrial and commercial buildings have flat roofs that are designed for snow loads. Rainfall detention ponding rings and gravel detention barriers are proven methods of retarding runoff at source and providing some storage. Design must make provision for emergency

overflow before water spills over the roof parapet or before the design loading limit for the roof is reached.

Parking Lot Storage - Large commercial and industrial parking lots can be designed to temporarily store several centimeters of water. Inconvenience to the public will be minimized if the water is stored in areas furthest from the building entrances. Alternatively, a shallow underground storage tank can be used to discharge to the storm sewers at a slow rate.

Porous Pavement - A third alternative, currently in the developmental stage, is the use of porous pavement with a subsurface reservoir of porous aggregate. In theory, where the ground below the reservoir is permeable, no runoff at all need occur from porous pavement except in the most extreme storm events. Porous pavement requires special maintenance to minimize clogging. Experimental parking lots using porous pavement have been constructed at the University of Delaware [12] and the Woodlands, Texas [13]. Porous pavement overlay has been successfully used by the Ministry of Transport and Communication on Highway #401 in Toronto to aid "drying" of the pavement surface during rains. The effect on the load capacity of the road base and frost heave effects have yet to be assessed [14].

Street Storage - If the number and size of catch basin inlets are limited, local streets can be designed to store up to 250 mm (10 inches), and arterial streets can store up to 150 mm (6 inches) of storm water for short periods with minimum inconvenience to the public. This is potentially useful in existing areas where the alternative is frequent basement flooding. Particular consideration must be given to hazards at roadway intersections, to the overland flow patterns that may result, and passage of emergency vehicles.

Surface Basins and Ponds - On-site detention basins which have been kept partially filled between storms can have the appearance of small lakes or park ponds. Aesthetic or recreational considerations are important in their design. These factors are discussed further in Section 5.3.

4.2.7 Controlling overland runoff during construction

The boundaries of a development project may have little relation to the original natural drainage patterns of the land. The planning process for development necessarily considers the integration of old and

new areas, but construction activity can create temporary patterns of overland drainage which are substantially different from those of the final developed land. The planning process for large-scale developments should consider and make provision for safe paths of overland travel for the major drainage system during all phases of development. Where necessary, a temporary drainage system, using open ditches (or similar drainage-ways) should be provided. The guiding principle should be protection of adjacent developed areas from property damage by misdirected overland runoff during development.

Without adequate controls, construction of storm water drainage systems may result in the transport of large amounts of sediment to watercourses during the development period.

4.3 Source Controls - Quality

This section deals with actions or techniques which can improve the quality of storm runoff by decreasing the availability of potential pollutants on urban surfaces. Materials washed from impervious surfaces are major contributors of pollutants in storm water, and source controls are a potential alternative in whole or in part to downstream treatment facilities.

4.3.1 Controllability of various sources

A number of reports [9,10,15,16] have indicated the potential polluting impact of vehicles, atmospheric dust and dirt fallout, salt, fecal material deposited by animals, and a large number of other activities, including littering by individuals. However, the relative contribution of the various primary sources and underlying mechanisms which lead to a particular rate of dust and dirt accumulation in an urban catchment are as yet poorly understood. The major pollutant from developing areas is sediment. Mitigative measures can greatly reduce the magnitude of soil erosion.

While source controls potentially extend to all primary sources, some primary sources are only controllable in an indirect sense - for example, reducing the extent of street littering by individuals is more an educational than a physical or legislative task. In other cases, even where there is a demonstrated environmental need for controls, the lack of a suitable alternative prevents the modification of urban activities to

totally eliminate the pollutant, e.g., the use of salt for deicing roads. In these cases, progress can only be made in the longer term through the actions of industry and higher levels of government. For example, the commercialization of lead free gasoline was a response by automobile manufacturers and the petroleum industry to an environmental need.

Municipalities, as part of their delivery of services, are responsible for depositing and removing various materials from road surfaces. These activities are carried out in an organized manner for specific purposes. There is considerable potential for upgrading or changing current practices where this is found desirable for environmental protection or enhancement.

4.3.2 Control location and objectives

From a practical viewpoint, there are two major categories of control locations at which pollutants may be intercepted: urban road surfaces, and the point of origin of pollutants (or closely adjacent to it). The former is normally of primary importance in developed areas, the latter in areas undergoing development. Table 15 lists objectives and methodologies for source control of pollutants. A discussion of specific control methodologies follows.

4.3.3 Erosion/sedimentation controls

The major source of sediment in urban runoff is construction activity. The design and planning process should consider the potential for erosion throughout the entire life of a development and ensure inclusion of appropriate mitigative measures.

The Province of Ontario and its municipalities consider erosion and sediment control as factors of indirect concern in land use planning. These factors are acknowledged through the assignment of appropriate zoning restrictions and by-laws. For example, hazard lands, such as ravine slopes, which are prone to erosion when the stabilizing effect of natural vegetation is removed, are generally classified as "open-space". The first level of planning should attempt to identify erosion-prone areas and so minimize erosion and consequent sedimentation through avoidance.

In Ontario, certain construction projects are subject to MOE review of the adequacy of proposed erosion-sediment control practices. Figure 29 is a schematic of the ideal planning process as it applies to

TABLE 15. OBJECTIVES AND METHODOLOGIES FOR SOURCE CONTROL OF POLLUTANTS

Objectives	Methodologies
1) To prevent interaction of pollutant with runoff water.	1) Use mitigative measures for control, e.g., erosion/sedimentation basins.
2) To prevent pollutant and/or contaminant runoff from entering the collection system (sewer) or receiving watercourse.	2) Remove pollutants with precipitation, e.g., snow.
	3) Control the application rate of potential pollutants, e.g., deicing salts, pesticides, herbicides, etc.
	4) Remove pollutants prior to precipitation, e.g., street cleaning.
	5) Enclose potential pollutants in weather proof enclosures, e.g., bulk salt storage facilities.

such projects. In principle, project plans should identify potential problems and their solutions prior to submission for approval to the MOE. As an aid to this process, the MOE has drawn up the document "Evaluating Construction Activities Impacting Upon Water Resources" [17]. Table 16 is a listing of site information needed to develop an erosion control program.

4.3.3.1 Controls during construction. Programs to control the loss of soils from construction sites should always be developed as an integral part of project planning, and should use an appropriate combination of erosion controls and sediment controls. The former are intended to minimize on-site damage caused by unnecessary or avoidable movement of soils by man or nature. The latter are intended to minimize the transport and loss of sediments from the site in runoff. For a residential subdivision, the active construction period can range to three years or more. During that period, sediment and a variety of other debris may find its way into the collection system as a consequence of site activities. Current storm sewer acceptance procedures in many municipalities involve

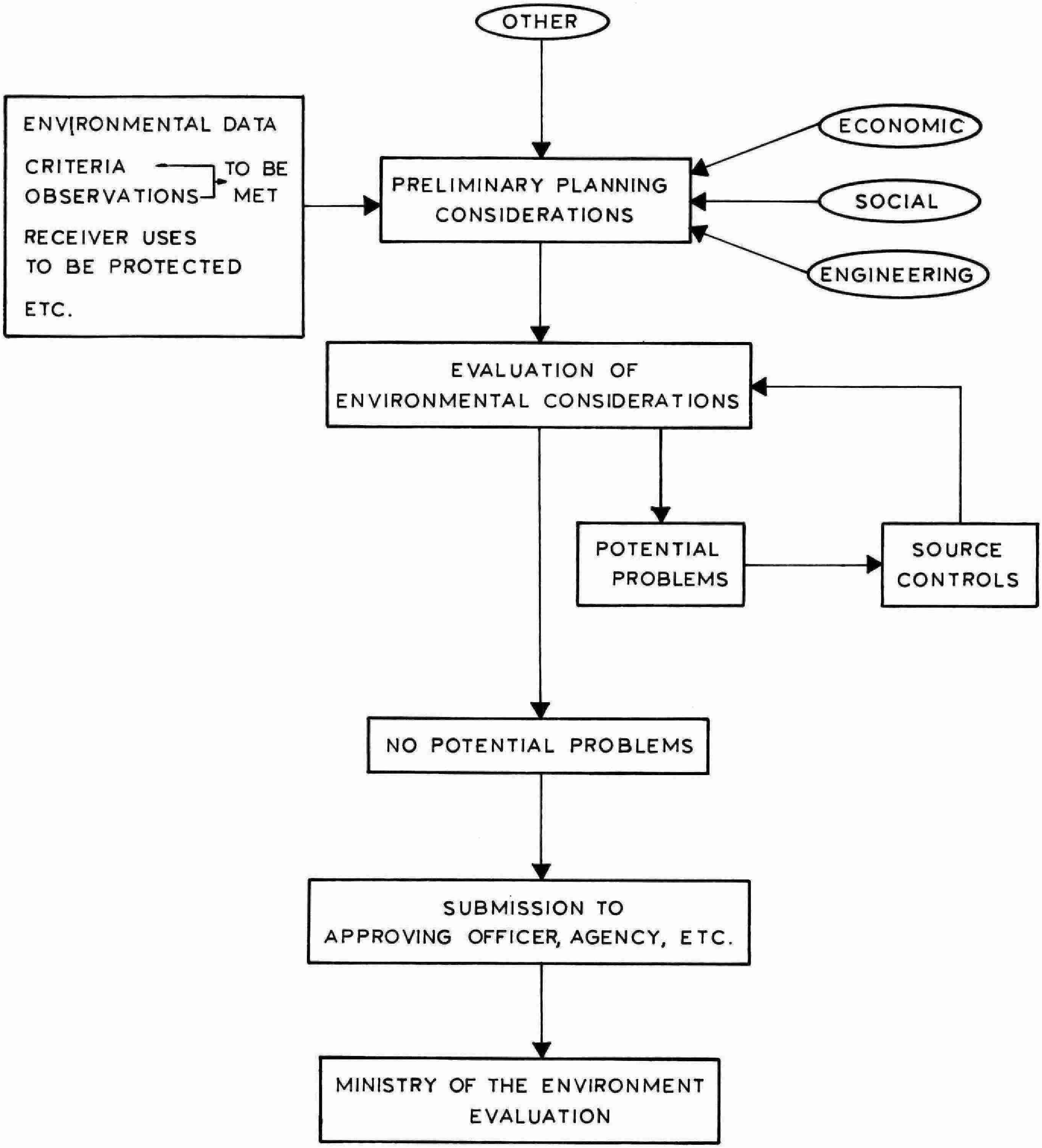

FIGURE 29. EROSION CONTROLS - THE PLANNING PROCESS

TABLE 16. SITE INFORMATION NEEDED TO DEVELOP AN EROSION CONTROL PROGRAM [17]

- Location of the site in relation to adjacent watercourses and drainage patterns.
- Water quality and uses within the adjacent watercourses.
- Extent of cuts and fills (e.g., dimensions, amounts of material to be placed or removed, grade of slope, etc.).
- Source (e.g., gravel pit, other construction site, etc.), type (e.g., rubble and sand, clean sand, gravel, etc.), and location of stockpiles for fill material.
- Proposed stream diversions or alterations.
- Grading plan - construction procedures and sequence.
- Location of wells, sanitary facilities, etc.
- Method of stream crossing (e.g., using a bridge, fording, etc.).
- Hydrologic data.

thorough flushing of new sewerage to remove accumulated sediment. A sedimentation pond placed during construction of the end of the new portion of the storm water collection system will intercept this accumulated material and prevent it being discharged into the receiver.

At present, while large-scale construction activities impacting directly on water resources are subject to review by the MOE, there are no universally applicable policies or guidelines in effect for smaller-scale projects, e.g., construction of individual subdivisions. Some other jurisdictions, the State of Maryland for example, have erosion-sediment control programs of much wider scope [18].

It is generally considered impractical to set universal standards or criteria for erosion-sediment control because of the variability in such factors as soil type, climate topography, vegetation, etc., which determine the rate of erosion, and because of differences in water quality and the uses to be protected within various streams. There is also no

single form of control measure that has the potential to completely eliminate the associated stream sedimentation. The soundest approach to effective control at construction sites is to use the best practical combination of demonstrated procedures and practices. The use of predictive tools such as the Universal Soil Loss Equation (Chapter 3) can also assist in the formulation of a sound erosion control plan.

Table 17 sets out five basic principles which should be followed in drawing up erosion-sediment control plans for construction sites. The Maryland erosion-sediment control program is based on consistent application of these principles. The state has produced a number of technical publications to aid in practical implementation of the program including a comprehensive workbook "Standards and Specifications for Soil Erosion and Sediment Control in Developing Areas" [19]. The program reported a 60-80% reduction in construction site sediment yields between 1966 and 1976 [20].

TABLE 17. TECHNICAL PRINCIPLES IN EROSION-SEDIMENT CONTROL

1) Plan the development to fit the particular site, i.e., a comprehensive land use plan.
2) Expose the smallest possible area of land for the shortest possible time.
3) Use erosion control practices to prevent on-site damage.
4) Use sediment control practices to prevent off-site damage.
5) Develop a thorough maintenance and followup operation.

4.3.3.2 Controls after development. If potential problem areas are recognized at the preliminary design stage, detailed design can include mitigative measures to minimize the need for maintenance (or remedial measures) after development is complete. Factors which should be considered include:

- Stability of drainage ways. These should be protected against scour by limiting the velocities of flow, applying protective layers (vegetation, rip-rap, etc.,) and ensuring adequate

maintenance to damaged segments. In instances where it is difficult to establish proper vegetative cover prior to a season of high precipitation, bared areas should be seeded and mulched.

- Suitability of outfall structure. In many cases, energy dissipators will be needed at the ends of outfalls to minimize scouring effects.

4.3.4 Snow removal

The major problem associated with snow removal is the introduction of deicing salts and sand to water courses, and a slow build-up of salt levels in the Great Lakes. Several studies have confirmed that chloride levels in watercourses can increase significantly as a result of deicing operations [10,21,22]. Additionally, accumulations of urban snow have been found to be highly polluted with various heavy metals.

The Ontario Ministry of the Environment in conjunction with the Ontario Ministry of Transportation and Communications and other agencies, has studied the pollution problems associated with deicing compounds and the dumping of snow. Studies are being conducted throughout North America to find suitable substitutes for deicing salts. In the interim, recommendations based on the MOE-MTC studies will help to minimize the problems associated with deicing salts. These recommendations are outlined in "Guidelines for Snow Disposal and Deicing Operations in Ontario" [23]. These guidelines strongly suggest that snow disposal to watercourses should be used only in emergencies. Alternative methods, such as disposal to carefully selected landfill sites, are favoured.

The use of snow melters, rather than snow removal, leads to accelerated runoff of deicing salts, suggesting that this practice should be used sparingly, especially where the receiver has sensitive uses.

4.3.5. Controlling chemicals on urban surfaces

The use of sand and gravel on roadways, instead of or in conjunction with reduced quantities of road salt, is one method of controlling the use of such chemicals. However, abrasives create different problems, such as excessive grit accumulation in combined sewers. As a result, there is presently no practical method of maintaining winter road safety which is completely satisfactory environmentally.

The use of chemicals such as fertilizers, herbicides, pesticides, etc., also adds to the pollutant load of local watercourses. The contributions of nutrients from fertilizers to the eutrophication of lakes has been well documented. The potential pollution risks associated with herbicides and pesticides have prompted the Ontario Government to require that users of such chemicals obtain a permit and be supervised in their application. Wherever commonly-used chemicals caused serious problems, restriction of their use or removal from the marketplace is a possibility.

Further studies are needed on many chemicals in current use, and alternatives should be sought for those that are detrimental to the environment.

4.3.6 Street cleaning

While most cities undertake some form of street cleaning for aesthetic reasons, emphasis has only recently been placed on street cleaning as a means of reducing the pollutant loading from urban runoff to watercourses. There is still a comparative lack of data on the cost-effectiveness of street cleaning.

Figure 30 indicates the great potential for pollution abatement shown in one particular study in the U.S. [24]. Another study [25] determined that the benefits of street sweeping included reduction of phosphorous levels in runoff. Over a short period, reductions of 43% and 29% were achieved in small test areas.

In U.S. studies, areas dominated by industrial land use experienced the greatest dust and dirt buildup, followed by commercial areas, and lastly residential areas. Techniques for quantifying dust and dirt accumulation are discussed in Chapter 3.

Significant amounts of material such as cement, aggregates, oil and grease, rubber, toxic metals, pesticides, grass, leaves, wood, etc., from various sources, and atmospheric fallout, accumulate on street surfaces. The pollution potential of these contaminants can be greatly reduced by removing the contaminants prior to rainfall. This removal process can be accomplished by modifying current street cleaning practices to pick up, in addition to the large size particles, the smaller factions which account for much of the total pollution load.

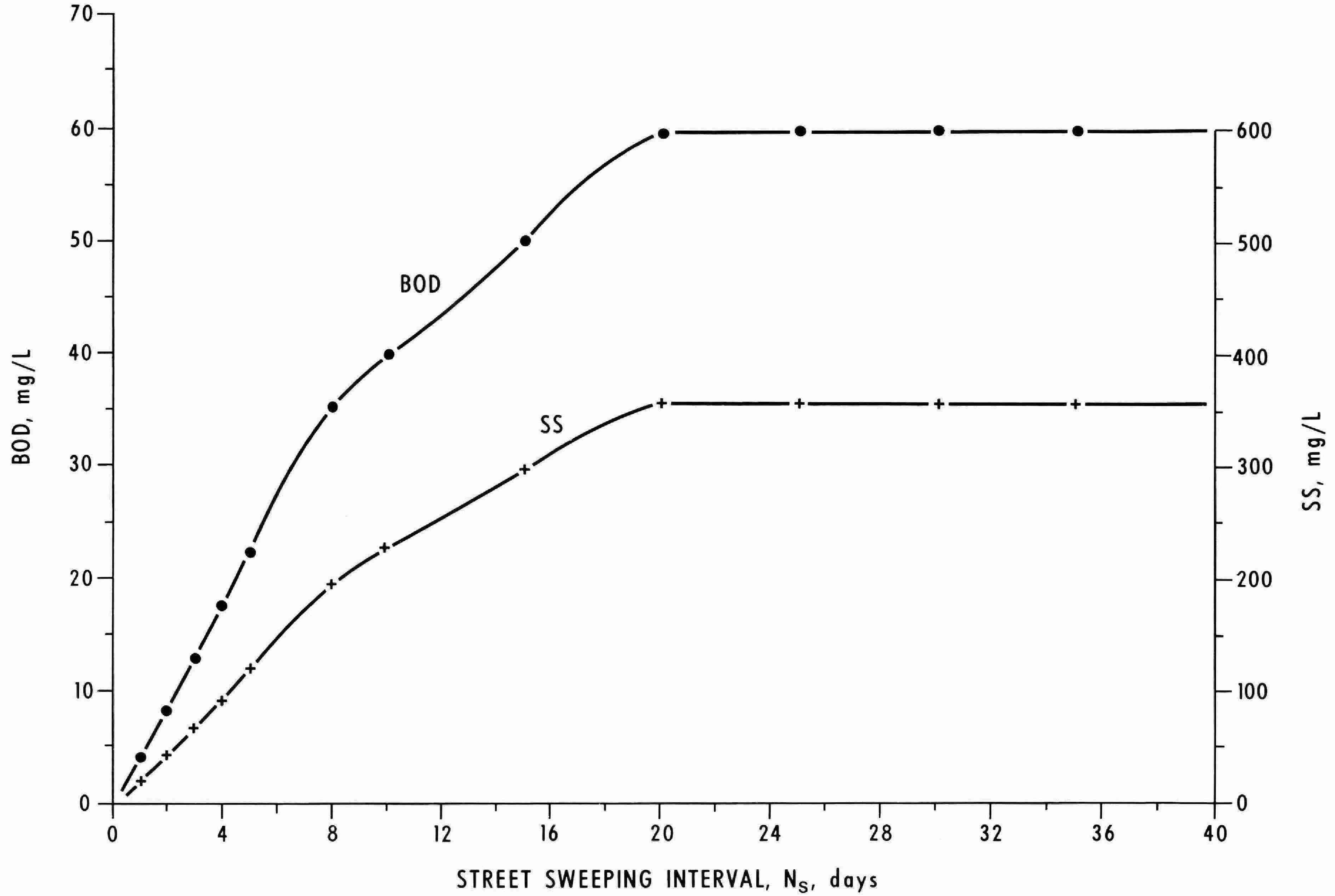

FIGURE 30. EFFECT OF STREET SWEEPING FREQUENCY ON ANNUAL BOD CONCENTRATION IN URBAN STORM WATER RUNOFF - DES MOINES [25]

Various studies [9] in the U.S. have shown that street cleaning effectiveness for pollutant reduction is a function of:

- sweeper efficiency,
- cleaning frequency,
- number of passes,
- speed of equipment,
- pavement conditions,
- equipment type,
- public participation and awareness.

Advanced cleaners on the market include broom and vacuum combinations which can achieve almost 90% efficiency if the other parameters are carefully considered to provide the maximum advantage to sweeper efficiency. Operator training is also very important and, in addition to being trained in the control of the machinery, operators should have some understanding of the importance of removal of fine particles.

The greater the frequency of cleaning between two rainfall events, the smaller will be the amount of material washed into urban drainage systems. The objective is to keep material buildup on road surfaces to a practical minimum.

It is very difficult for the majority of sweepers to achieve maximum pick-up efficiency in one pass. The greater the number of passes, the greater the amount of fines that will be removed.

Most equipment is designed to provide maximum efficiency at a certain operating speed. If this speed is exceeded, the sweeper efficiency will fall significantly.

Holes in a road surface are hard-to-reach places for sweepers. In addition, pavement deterioration continually adds cement and aggregate material to the pollutant loads on the surface. Road surfaces should be adequately maintained.

Parked cars are major obstacles to efficient cleaning. The public should be well informed on the need for cleaning and the need for streets to be clear of parked vehicles so that effective cleaning can be accomplished. The public should also be informed of the contributions individuals can make to reduce the amount of material that ends up on a road surface.

4.3.7 Other methods

Municipalities, directly or through by-laws, control the removal of household garbage and the wide variety of domestic and industrial solid wastes generated in urban areas, including fallen leaves, grass clippings, and other "harvested" vegetation. Effective provision of these services is an important part of pollutant source controls methodology.

The physical enclosure of pollutants assumes particular importance in the storage and transport of fine bulk solids. Use of storage domes for road deicing materials, as practised by the Ontario Ministry of Transportation and Communications, represents a practical application of this technique.

4.4 Natural Engineering of Storm Water Drainage Systems

In newly developing areas, the use of "natural engineering" principles can radically alter the form and appearance of the storm water drainage system compared with that designed by traditional methods. These principles are potentially applicable to any large scale development, as an integral part of overall planning.

The "natural engineering" approach seeks to take maximum environmental advantage of the development process. Techniques which closely duplicate or enhance the original drainage pattern of the site are used, in order to enable runoff to be managed as a resource for aesthetic and recreational enjoyment.

Land is developed in such a manner as to minimize overall impact on water resources, with the more intensive land uses generally directed to those locations where potential impact is least. The need for, nature, and extent of mitigative measures is decided individually for each area, utilizing on-site measures whenever possible. Recognition is given to the desirability of preserving aquifer recharge areas, and ensuring that design of the drainage system assists in maintaining groundwater levels and/or base-line flows in streams at desired levels.

The basic principles involved are widely applicable although the details of implementation will vary from site to site and may be influenced greatly by climatic conditions, topography, soil types, land cost, etc. The handbook "Water Resources Protection in Land Development" [13] discusses the principles and practical aspects of a wide variety of on-site and off-site control measures for natural engineering applications.

An outstanding example of a development planned according to natural engineering principles is the Woodlands near Houston, Texas [13,26].

Some recent housing developments in Winnipeg have utilized one of the principal features of natural engineering - interconnected lakelets for storm water detention. These lakelets largely replace the trunk storm sewers that would otherwise be needed (Chapter 5).

4.5 Quantity and Quality Control Applications

This section describes three Ontario applications of some of the control techniques described in this chapter.

Source Controls at Merivale Industrial Colony [27]. The Merivale Industrial Colony occupies 80 ha (200 acres) in the 525 ha (1300 acre) Merivale watershed. The watershed is located in the City of Nepean, and drains to the Rideau River by a tributary. A study carried out prior to construction of commercial establishments proposed two forms of storm water management - source controls and in-line storage.

The source controls were designed to promote on-site sedimentation of pollutants from storm water; the pollutants would then be removed from the impervious areas by brush or vacuum sweeper. Controls will also provide some on-site runoff attenuation. A number of development control agreement clauses relating to the techniques proposed for the management of runoff were included in the site plan agreements, and then registered on title. The agreements include clauses relating to erosion control during construction, and the design and maintenance of facilities affecting storm water pollutant loads. The municipality was also allowed reasonable access to private property in the colony, to remove sediments.

A set of design guidelines were prepared and presented to the developer prior to detailed site design. The design guidelines encompass land grading, roof ponding, and catch basin and rainfall design data. The purpose of these guidelines was to ensure proper planning and reduction of storm water pollution. Analyses undertaken to determine the effectiveness of such control measures indicated at 14.1% reduction in BOD_5 and 12% reduction in SS for the entire watershed from a one-year synthetic storm.

Storm Drainage Detention Ponds - Upper Canada Mall, Newmarket, Ontario [28]. This shopping centre is located close to the intersection of Highways 9 and 11 in the Town of Newmarket. The site is approximately 23 ha (57 acres) in area, of which 12 ha (30 acres) are developed with department stores, small stores in a mall, and surface parking. A further 3 ha (7 acres) are occupied by a park and storm water detention lagoon.

The municipality provided conventional water and sanitary sewer services to the centre. However, there was no storm drainage outlet in the vicinity with sufficient capacity to handle an increase in storm water runoff from the site. An on-site storage facility was therefore necessary for peak flow control. Detention lagoons were built as an integral part of an on-site park.

The lagoons, already in operation, were designed to discharge at a peak rate not exceeding that from the site prior to development. A manually-operated sluice gate can be used to give some adjustment in the discharge rate to the storm sewer, which is limited to 0.5 m^3/s (20 cfs). A fountain, installed in each lagoon, is designed to provide a limited level of aeration and to break up the surface of the water to mask evidence of traces of oil that may run off from the parking area. The lagoons have the capacity to store runoff from a 25-year storm without over-topping the surrounding landscaped embankments.

For safety, the slope of the lagoon banks and bottom is no more than 5:1 and normal water depth does not exceed 0.6 m (2 ft), rising to 1.4 m (4.5 ft) during the five-year design storm.

Figure 31 shows the major features of the drainage system along with the location of the ponds.

Storm Water Detention Tanks for Parking Lot Runoff - Borough of York [29]. The Borough of York in Metropolitan Toronto has a large number of underground storage tank facilities (cisterns) under parking lots. These have been installed in combined sewer areas in an effort to minimize local basement flooding.

The cisterns, installed routinely since 1957, follow specific design standards enforced by a municipal by-law. Design criteria include the following:

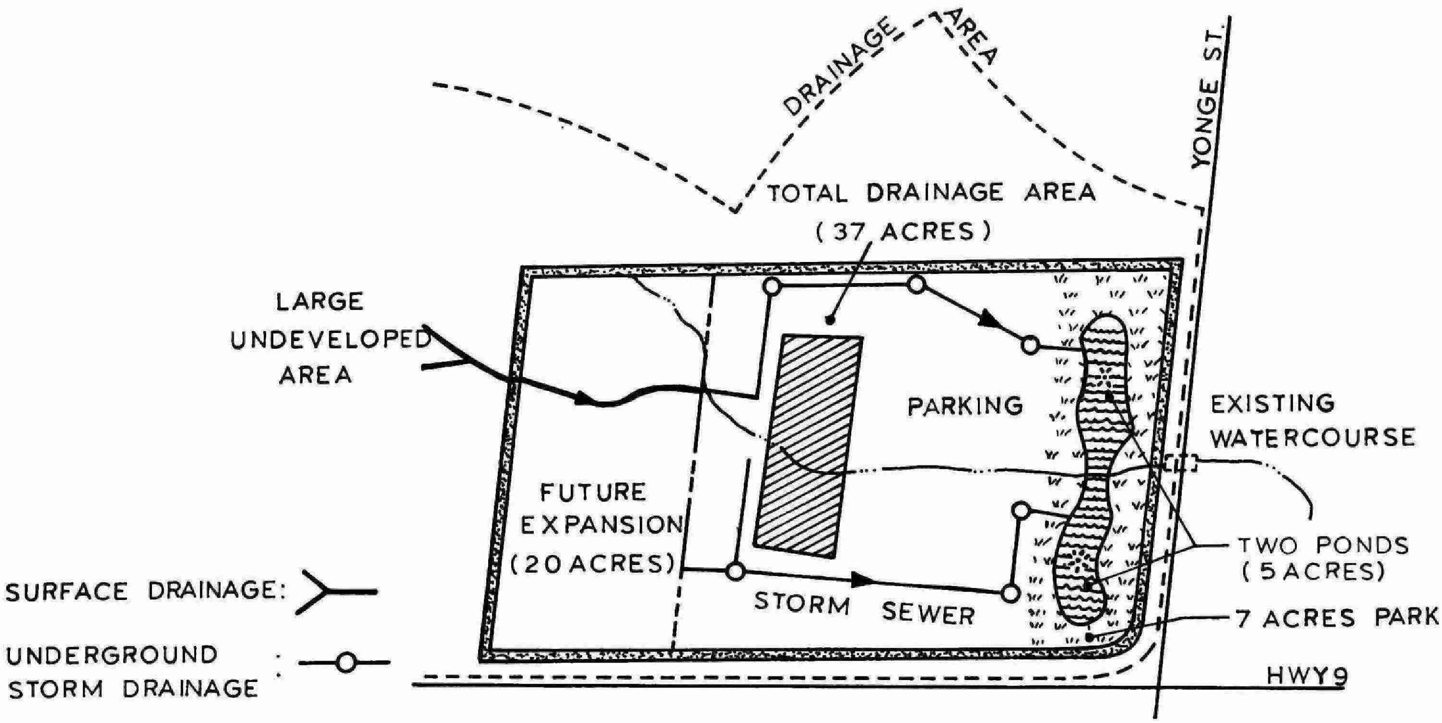

FIGURE 31. STORM WATER DETENTION POND - UPPER CANADA MALL, NEWMARKET, ONTARIO [28]

- Parking lots in excess of 230 m^2 (2500 ft^2) that are connected to the municipal sewers should have an underground storage tank of steel or reinforced concrete, with storage capacity of 1.8 $m^3/100\ m^2$ (60 $ft^3/1000\ ft^2$).
- The discharge at the outlet from the storage tank is controlled by limiting the size of discharge pipe. Sizes range from 20-25 mm (3/4"-1") in diameter depending on the area.
- All catch basins upstream of cisterns must be provided with gas traps.
- Figure 32A shows the installation of a typical detention tank in a parking lot.
- Figure 32B gives the side elevation of the sump
- Figure 32C is the side elevation of the storage tank.

The requirement to route storm water through a cistern often necessitates pumping to raise the water to a sufficient elevation for flow to the street sewer.

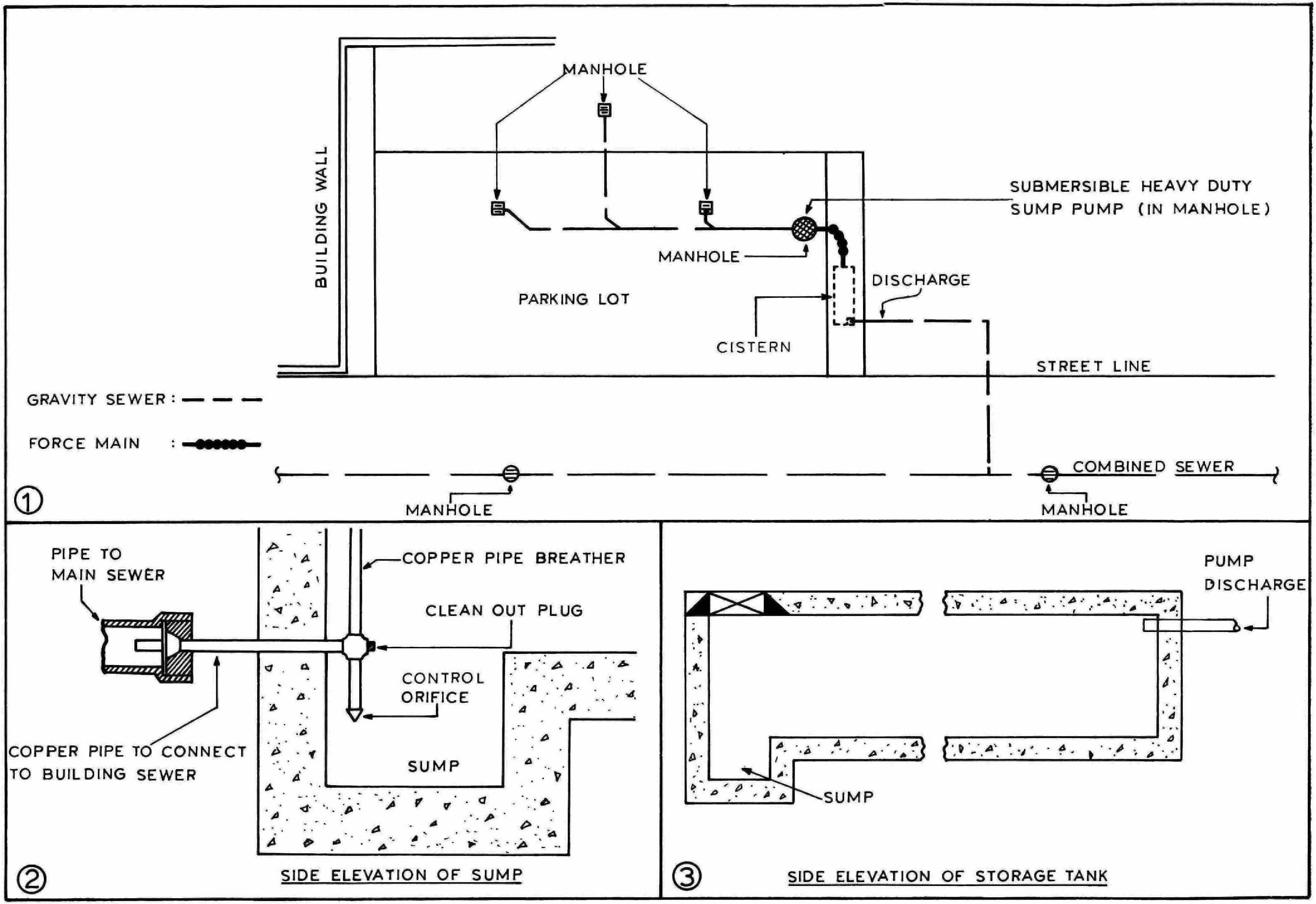

FIGURE 32. TYPICAL PARKING LOT CISTERN FOR STORM WATER STORAGE - BOROUGH OF YORK, METROPOLITAN TORONTO [29]

Very few problems have been reported. The borough provides advice on initial cleanout of storage tanks, which is carried out by private contractors with catch basin cleaning equipment.

More recently, the borough has insisted on roof top detention on commercial buildings as a further measure of runoff control.

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5 COLLECTION, STORAGE AND TREATMENT ALTERNATIVES FOR URBAN DRAINAGE PROBLEMS

5.1 Introduction

This chapter deals with control techniques applicable to the collection, storage and treatment of urban runoff to meet desired objectives.

Reasons for seeking control at a given point in the drainage system most commonly include property protection, maintaining or enhancing receiving water quality, and cost-effective design of drainage and treatment facilities. While control techniques can be grouped and described according to their similarities, individual techniques may not be applicable in both storm and combined sewerage systems. The approach taken has been to identify the applicability of various techniques within each group as they are discussed in the text.

Collection system controls are those which can be used during interception or conveyance of storm water or combined sewage within the piped elements of the drainage system. Section 5.3 contains information on various techniques which can be used to meet the space requirements of storm water or combined sewage between entry to the collection system and discharge to a receiver. Sections 5.2 and 5.4 contain further material on specialized aspects of storage.

Treatment refers to those actions or techniques which can remove pollutants from storm water or combined sewer overflow (CSO). The term pollutant refers to dissolved matter, suspended matter, or microorganisms which must be removed to protect, maintain, or enhance receiving water quality.

Each section contains examples of the control techniques cited, drawn where possible from Ontario experience.

5.2 Collection System Controls

5.2.1 Collection system characteristics

By definition, a separated sewerage system carries sanitary wastewater and surface runoff in separate conduits Sewer use by-laws address the quality of industrial wastewater which may be discharged to either type of sewer. In general, non-polluted flows, such as once-through cooling water, may be discharged to storm sewers, while polluted flows are discharged to sanitary sewers.

A combined sewer carries both sanitary wastes and surface runoff in a single conduit.

In most urban areas the collector system was originally based in principle upon either the separated or combined system. However, today many systems defy precise categorization; the cumulative effects of planned or haphazard modifications of sewerage, the installation of temporary relief sewers from one type of sewer to another, and similar measures, have often changed the system greatly. Table 18 [1], while not exhaustive in scope, lists the more common variants of sewerage found in Ontario.

Flows in the "partly combined" and "interconnected storm and sanitary" collector systems identified in Table 18 may respond to precipitation in much the same manner as a fully combined system, although peak flow rates may be lower. Accordingly, control techniques applicable to fully combined systems may also be applicable in these two types of systems.

The original reasons for construction of separated rather than combined sewers included:

- a reduction in health hazards associated with the discharge of CSO, especially discharges to small or intermittent streams and watercourses; and
- a reduction in transmission costs, since open ditches could be used for conveyance of storm water.

Separation was later seen to offer further advantages with respect to the consistent treatment of sewage in both dry and wet weather. Later yet, separation of existing combined sewers was seen to offer immediate relief from basement flooding and generally better pollution control.

With the passage of time, several recurrent problems have become apparent in separated systems. For example, in many cases, the sanitary sewage per person flows are much higher than originally anticipated due to extensive inflow-infiltration. As a result the true peaking factor, including sanitary by-pass, may be much higher than predicted by design. Additionally, seasonal variations in sewage flow and strength may be

TABLE 18. CLASSIFICATION AND CHARACTERISTICS OF COLLECTION SYSTEMS [1]

Features	Sewer Type: Combined Sewers	Partly Combined Sewers	Interconnected Storm and Sanitary	Separated Storm	Separated Sanitary
Flow mixtures at peak flow	1 part sanitary in 50-70 parts storm flow.	1 part sanitary in 10-50 parts storm flow.	1 part sanitary in 5-20 parts storm flow.	Storm flow with traces of sanitary flow.	Sanitary peak flow 2 to 4 times DWF.
Flow sources	All storm and sanitary drain connections	Part of storm flow, mainly from catch basins, collected in separate "street sewers". Sanitary flow plus balance of storm flow collected in partly combined sewer.	Adjacent separate sewers interconnected intentionally for relief or incidentally, e.g., by major leakage and infiltration.	Except for the odd incorrect sanitary drain connection, storm sewer laterals are connected only to catch basins.	Sanitary flow dilution limited to normal incidence of inflow and infiltration.
Other Characteristics	Combined trunk sewers may receive flow from upstream sanitary or storm sewers.	Sometimes original storm sewer receiving sanitary or combined flow from upstream.	Interconnection in system may include sanitary and/or storm sewers draining into combined sewers.	Storm flow only, draining to open water by gravity.	Sanitary flow conveyed to sewage treatment, nearly always includes pumping somewhere.

precipitation or snowmelt-dependent. These factors present real difficulties in ensuring consistent, efficient performance of secondary or advanced wastewater treatment facilities. In large measure, inflow-infiltration problems stem from deficiencies in older sewers. There is no inherent reason why inflow and infiltration should not be held within reasonable limits with present day design and construction practices. However, resolving existing problems can be a difficult and expensive task.

In many urban areas, storm drainage has been routed through small watercourses which act as flood control channels and ultimately discharge to a major receiver. Indiscriminate use of these channels and poorly located outlets have often eroded 'unimproved' sections severely. The hydrologic and other changes which occur with urbanization can result in channel erosion and a continuing decline in water quality, leading to loss of the entire stream as a natural watercourse, and ultimately to the need for channelization or even enclosure.

Storm sewer systems offer direct access to watercourses, and effective policing of "fugitive" discharges during dry and wet weather is necessary. This can be difficult because of the multiple points of discharge. Emergency spills from industries and from other sources can also enter watercourses readily.

The pollutant loadings from landwash and the quantity of storm water increase in an absolute sense as land use intensifies, and storm water runoff can become a significant source of pollutants. When comparisons are made of annual loadings of pollutants in "effluent" from treated sewage and storm water from the same area, the relative contribution of the storm water becomes larger each time the efficiency of sewage treatment is increased. Storm water conveys 'exotic' pollutants such as heavy metals, asbestos, and pesticides directly to receiving waters. Evaluation of the sources and pathways by which exotic pollutants reach receiving waters and their long-term effects is in the developmental stage and relative contributions from landwash are as yet unclear. As knowledge grows it is quite possible that effective control of certain parameters will hinge on control of storm water quality from certain land uses, prompting wider-spread use of 'end-of-pipe' or 'source' controls.

The question is frequently raised as to whether some form of combined sewer system, making extensive use of storage and other means to control and limit pollution from overflows, may offer advantages over a separated system. A combined system of this type is presently hypothetical and would seem to be attractive only at high land use intensities. Since the core areas of many cities are already served by combined sewers, 'total system management' of the existing combined sewers is worthy of serious consideration as a pollution control alternative to sewer separation.

In new development, the advantages of separated sewers still outweigh the disadvantages, especially at low and medium land use densities. For example, the sanitary component in combined wastewater necessitates extensive treatment before the waste can be considered a resource for aesthetic or recreational enjoyment, but separated storm water runoff from new low and medium-density developments requires minimal upgrading for many beneficial uses.

Intensified land use in separated sewerage areas may gradually change runoff characteristics such that storm water controls become essential to maintain receiving water quality. In principle, various combinations of upstream storage, source controls, downstream storage, and end-of-pipe treatment can be used. At the present time, there is little information available to aid in deciding which approaches are most cost-effective for application at high densities. There is also no convincing generalized approach available for decision-making or evaluating alternatives - each case must be evaluated individually.

5.2.2 Sewer system physical survey, inventory and inspection

In approaching the total management of a combined sewage collection system, it is necessary to define the system accurately in a physical sense and determine its present condition. The extent and degree of detail to which this should be done is a function of the management program objectives.

Surveys of various types are required to permit:

- hydraulic simulation modelling,
- real-time control,
- sewer maintenance and/or rehabilitation programs,
- inflow-infiltration analysis.

The minimum data requirements consist of:

- basic mapping of the entire system,
- inventory of major sewer lines and appurtenances,
- physical inspection of major sewer lines and appurtenances.

5.2.2.1 Map data. Map data should include at least the following basic items:

- size of sewers and direction of flow,
- type of sewer,
- locations of treatment facilities, pumping stations, regulators, by-pass points, and watercourse crossings.

An indication of all manholes and their inverts, and sewer slopes is desirable, although not essential for all purposes. For systems where maps are available, map data should be confirmed and updated prior to field verification. In general, a map scale of 1:4800 is desirable.

Where no map data exists, maps should be prepared. A description of the methodology of sewer map preparation is contained in a recent EPA publication [2].

Once a sewer map is prepared, it may be augmented with other pertinent data. The following information may be indicated or overlayed on the sewer maps:

- topography of the study area,
- soil formation,
- depth of the groundwater table,
- sewer age,
- land use and population distribution,
- known problem areas by problem category.

The addition of this data to a sewer map can give indications of sewer system deficiencies, potential problem areas, and the interaction of hydrologic features and surface activity with the sewerage system.

5.2.2.2 System inventory. An inventory of the sewerage system should be undertaken in conjunction with map preparation. A sample form showing the type of information needed and suitable means of organizing an inventory

is presented in Table 19 [2]. The Ontario Ministry of Transportation and Communications (MTC) has developed a method of inventorying and classifying municipal sewerage and evaluating deficiencies and future needs. Their approach is described in a comprehensive procedures manual [3]. In the MTC approach, sewer needs are evaluated simultaneously with other servicing needs, particularly road repairs. Replacement or repair of sewers can then be more readily integrated with road improvement, and can be perceived as a logical and necessary activity in improving and maintaining an adequate municipal infrastructure.

5.2.2.3 System inspection. An inspection of the physical condition of system elements should be undertaken concurrently with system mapping and the production of an inventory. The following data should be recorded:

- cracked or broken sewer lines,
- dislocated joints,
- root intrusions,
- cracked or leaking manhole structures,
- sewer lines with debris or deposits,
- improper connections,
- sunken manhole covers, and
- corroded piping.

Visual inspection is the principal means of assessing sewer system physical condition. Major sewer lines and system appurtanances are often sufficiently large to allow entry and direct visual inspection. Inspection of smaller lines may be conducted by "lamping" or where necessary by television or photographic inspection.

"Lamping" employs a powerful light source directed through the sewer line from one manhole to the next. A mirror held at the mouth of the incoming sewer in the downstream manhole reflects the light and allows visual inspection.

Television or photographic inspection both employ a camera pulled by cable between manholes. In the case of TV inspection, the sewer interior is displayed immediately on an external viewing screen. Photographic inspection requires film processing before results can be assessed.

TABLE 19. TYPICAL DATA SHEETS FOR THE INVENTORY OF A SEWER SYSTEM [2]

I. SANITARY GRAVITY SEWERS OR COMBINED SEWERS

Pipe Age	Size	Length	Pipe Materials	Joint Type & Material	Number of MH's	Sewer Depth	Bedding Material	Backfill Material
Sub-Total								

II. SANITARY FORCE MAIN

Pipe Age	Size	Length	Material
Sub-Total			

III. OVERFLOWS

No.	Location	Description	Frequency of Over-flow	Over-flow Rate	Probable Causes of Overflow	Discharge Point

IV. BY-PASSES

No.	Location	Description	Frequency of flow by-pass	By-pass Flow Rate	Condition Required for By-passing	Discharge Point

V. PUMPING STATIONS

No.	Location	Type of Pumps	Pump Rate	Average Daily Flow and Low Groundwater	High Groundwater

Both TV and photographic inspection are expensive and require sewer pre-cleaning. They are best employed after a systematic, preliminary screening procedure has identified those sections of pipe likely to need more detailed inspection. Various factors, including sewer age and the frequency of flooding complaints, may be employed as preliminary indicators of such sections.

5.2.3 Maintenance of combined sewer systems

A comprehensive maintenance program is an integral element of combined sewer management. The functions of such a program include:

Preventive maintenance - intended to ensure continued efficient operation and the protection of capital investment.

Intelligence - intended to provide ongoing detection and cataloguing of deficiencies as they arise.

Repair - intended to eliminate minor system deficiencies. Repair activities are performed in response to an observed deficiency or complaint.

Different system elements require specific maintenance procedures. For maintenance purposes sewer system elements may be categorized as follows:

- sewers (i.e., piping),
- appurtenances (manholes, catch basins, and inlet/outlet gratings), and
- control and pumping structures.

Sediment and other accumulated materials in sewers decrease the pipeline capacity by reducing the area available for flow, and by increasing frictional losses. A systematic, routine program of sewer cleaning is an effective counter-measure and is of particular importance in the management of combined sewer systems to limit unnecessary overflows.

The materials commonly found in sewers include:

- tree roots,
- grease,

- sediment,
- mineralization products, and
- trash.

In some cases the greatest benefit may be gained from a program of control on the ingress or input of these materials. Studies have indicated, for example, that chemical root control produced better results than after the fact removal by cleaning [2].

Sewer deposits are more pronounced in areas where large quantities of abrasives are applied to streets during winter. In these areas, an effort should be made to evaluate and review street deicing and cleaning practices in relation to sewer maintenance.

Cleaning methods are employed on a manhole to manhole basis. In any cleaning operation there are five tasks. They are as follows:

- locate and obtain access to manhole(s),
- dislodge material,
- transport material,
- remove material, and
- dispose of material.

It should be noted that the ultimate disposal of materials removed from sewers may be costly. The means of disposal is generally by landfill, and suitable sites may be remote from the cleaning location.

Four types of equipment are available to dislodge and transport material in sewers, for light to heavy duty applications, respectively. Table 20 presents a summary of the principle types of cleaning equipment, and debris-removing devices.

Manholes, catch basins, and inlet and outlet grates all require regular inspection, cleaning, and repair. When sewer line cleaning is performed, manhole inspection can be conducted and deficiencies may be scheduled for repair. Some aspects of catch basin cleaning are discussed in Section 5.2.7.

Automated combined sewer regulators and pumping facilities require a maintenance approach in keeping with their electro-mechanical character. Both static and automated regulators should receive routine

TABLE 20. EQUIPMENT FOR SEWER CLEANING AND DEBRIS REMOVAL [2]

Type	Mode of Operation	Size Range	Most Effective	Least Effective
Rodding Machine	- power driven flexible rods with various end attachments -- augers, cork-screws, root saws, etc.	15-38 cm (6-15")	dislodging roots and blockages	heavy solids transport
Bucket Machine	- bucket pulled between two manholes by winched cables	20-90 cm (8-36")	dislodging, transporting, and removing sand, gravel, bricks, rock, roots	cannot be used in structurally damaged pipe
High Velocity Water Machines	- cable-drawn high velocity nozzle arrangement	15-40 cm (6-16") or larger	dislodging and transporting sludge, mud, sand, and gravel	large debris
Hydraulically Propelled Devices	-sewer ball, cone kite - depend upon water pressure for cleaning force and propulsion	15-90 cm (6-36")	dislodging sludge, mud and sand	blockage
Debris Removing Devices	- removes cleaned material from sewer. Trash pumps and vacuum machines commonly used.			

inspections followed by any necessary maintenance. Automated regulators should also be tested frequently, to ensure that efficient operation is maintained.

5.2.4 Rehabilitation of combined sewer systems

Sewers, like most structures, deteriorate with time. Life expectancy commonly ranges between 50 and 100 years after which time they may no longer be structurally adequate to contain the flows within the conduit or to sustain the externally imposed ground loadings. The combined sewers within a total system usually have greater average age than the separated sanitary sewers, and hence may have more urgent need of ongoing rehabilitation programs. Timely action can avoid a crisis situation in which large capital sums must be committed to rehabilitation over a short time span. Objectives of rehabilitation may be stated as follows:

- to preserve pipeline and appurtenant structures in order to extend their useful life and capability to withstand the effects of age, erosion, corrosion, settling and loading;
- to correct existing structural deficiencies and impending structural failure from various causes;
- to reduce or eliminate infiltration of groundwater, exfiltration of sewage and, in some instances, inflow of surface waters;
- to recover pipeline hydraulic capacities.

Those collection system deficiencies which most often indicate the need for rehabilitation are listed below. Causes of the deficiencies can range from inadequate design or construction to old age.

- cracked pipe,
- broken or crushed pipe,
- improperly supported pipe and pipe with improper line and grade,
- deteriorated pipe (erosion/corrosion),
- deteriorated mortar joints in brick pipe and manholes,
- leaking pipe joints in street sewers,
- leaking building sewers,
- leaking manhole external drops,
- leaking or deteriorated manhole walls, bases, and troughs,
- leaking or deteriorated wet wells, lift stations, regulators, and flap gates.

Rehabilitation may be achieved by the following three principal methods:

- excavation/replacement,
- chemical grouting,
- pipe lining.

Detailed discussion of rehabilitation techniques is beyond the scope of this manual. An EPA publication [2] gives extensive descriptions and costing methods for sewer rehabilitation.

5.2.5 Maximizing in-system storage

In both storm and combined sewers, the storage capacity of the sewer system itself can be used to lower the peak rate of runoff. In combined sewers, this may reduce the number of overflows by permitting diversion of a higher total volume to an interceptor, for downstream treatment.

Both types of sewer are typically designed for peak flows from a two to ten year storm and have excess storage capacity whenever the flow rate upstream of a given point is less than the peak design flow (see Figure 33).

Adjustable or fixed geometry devices to control the rate of outflow from a catchment enable storage in the upstream reach. Adjustable devices are usually designed to respond to a level signal in the conduit at a locally or remotely actuated control station. Both variable and fixed geometry devices must be designed such that some critical upstream level is not exceeded. The critical level may occur in the sewer which contains the control device or in a tributary sewer. Where surcharging in trunk sewers must be avoided, flat grades over long reaches are essential to permit accumulation of significant volumes of upstream storage.

In large combined sewer systems the utilization of storage can be further enhanced by the use of flow routing. Flow routing seeks to minimize overflows by making as much use as possible of the available storage in those portions of the sewer system receiving the least volumes of runoff, and to utilize any storage available in the interceptors. A supervisory control system is essential before a sewer system can be operated with flow routing.

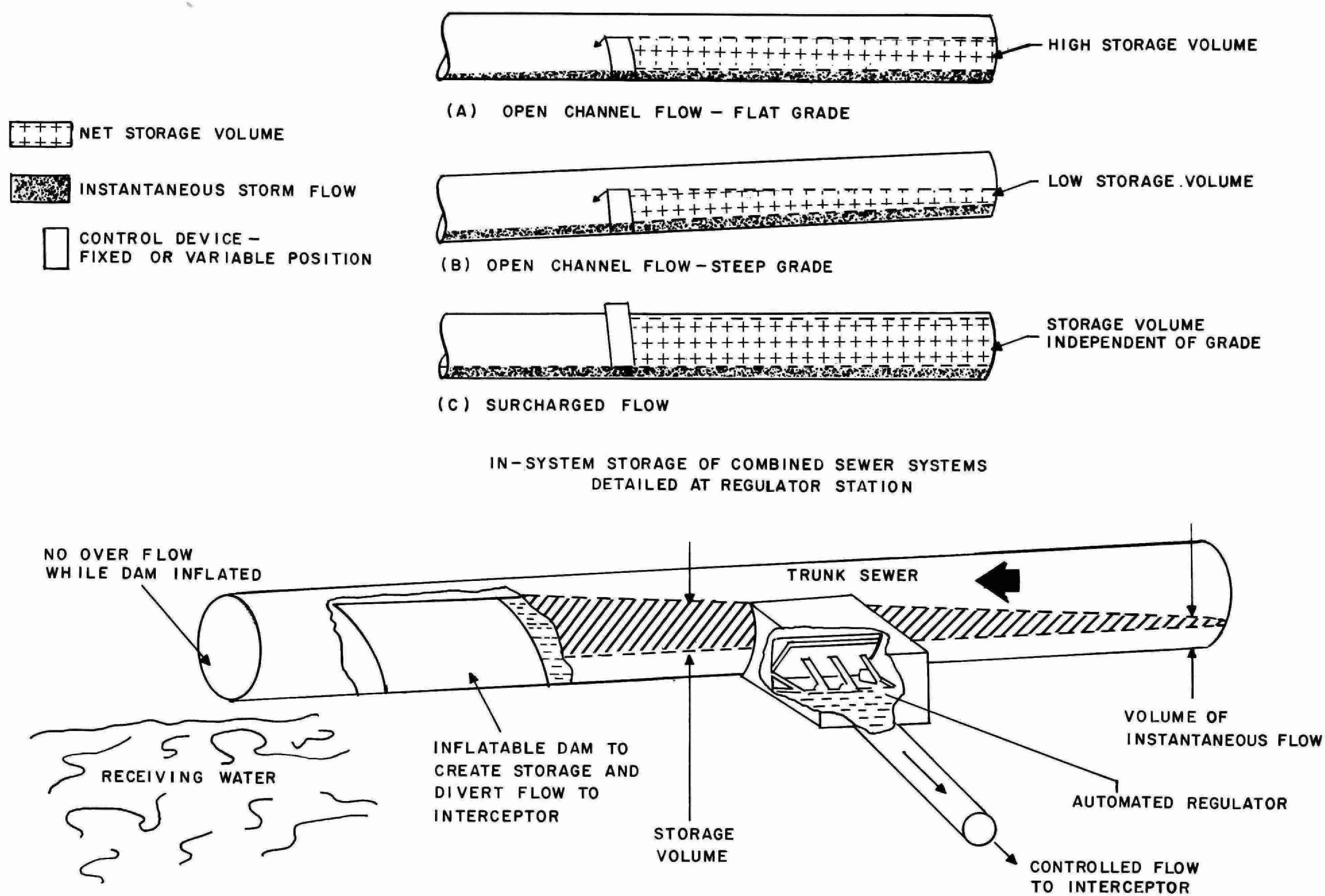


FIGURE 33. SCHEMATIC REPRESENTATION OF IN-SYSTEM STORAGE

Unwanted sedimentation is a potential problem with in-system storage. Velocities below 0.6 m/s (2 ft/sec) usually cause graded sedimentation with coarse deposits occurring upstream and finer deposits downstream. Various approaches have been used to prevent excessive accumulation, including flushing, and hydraulic and mechanical cleaning.

In combined sewer systems, supervisory controls can be coupled with the use of routing controls and off-line storage to reduce the frequency of overflow.

The simplest approach requires monitoring of rainfall, and rates of flow and critical levels, in addition to signal telemetry. A central computerized display console and data logging are needed to provide operator assistance in decision-making. Automated regulator stations enable selection and control of overflow locations.

More sophisticated control systems can include a variety of features such as:

- combined sewer overflow quantity monitoring,
- receiving water monitoring,
- predictive capability on storm movement,
- automatic operational real-time control,
- quality monitoring.

Some form of supervisory control system will be needed whenever total management of a large combined system is chosen as a pollution control method, to provide data on the status of in-line and off-line storage, pumping stations, etc.

A relatively simple supervisory control system, equipped with effective data logging, will accumulate data which can be used subsequently in hydraulic simulation to analyze the effectiveness of further pollution control measures in reducing overflow. The accumulated data is especially useful if coupled with a sampling program to monitor overflow quality.

Additionally, the use of a telemetry system and centralized 'status' information on sewer system operation permits early identification of unusual system operating conditions which might lead to unnecessary overflows.

Figure 34 [5] shows two types of remotely actuated and controlled regulator stations which have been used as part of in-line storage and routing control systems. In each case, the complete station includes two devices which together control upstream sewer level, volume of overflow, and volume intercepted. Total flow going in each direction may be measured directly, or estimated from prior calibration of the stage-discharge relationship of the control device. In automated regulator stations overflow "gate" positioning must be positive and swift to avoid unsafe upstream operating levels.

5.2.6 Various measures to reduce overflow volume

The following group of control techniques can be used to reduce the frequency or gross volume of combined sewer overflows. The order of presentation reflects likelihood of applicability in typical combined sewer systems - inflow controls are potentially the most widely applicable.

5.2.6.1 Reducing inflow. Table 21 shows the most common sources of inflow. Most of the private property sources have been discussed in Chapter 4 under "source controls". The exception is cooling water from industrial or institutional sources; the volume and acceptibility of such water is regulated by sewer use by-laws.

Identification of inflow sources is generally technically easier and less costly than identification of infiltration sources. However, removal of inflow sources may present significant difficulties for municipalities in terms of community acceptance. For example, where connection of roof leaders and the like was not forbidden by by-law at the time of original construction, municipalities are usually reluctant to pass and enforce retroactive by-laws which may cause householders considerable expense and inconvenience. In some cases arranging for alternative means of disposal may be technically difficult due to small lot size, excessive paved area, etc. These difficulties do not alter the general principle that disconnection of inflow sources should be implemented whenever possible.

The collection system inflow sources listed in Table 21 include some related to a primary function of combined sewers - namely, surface

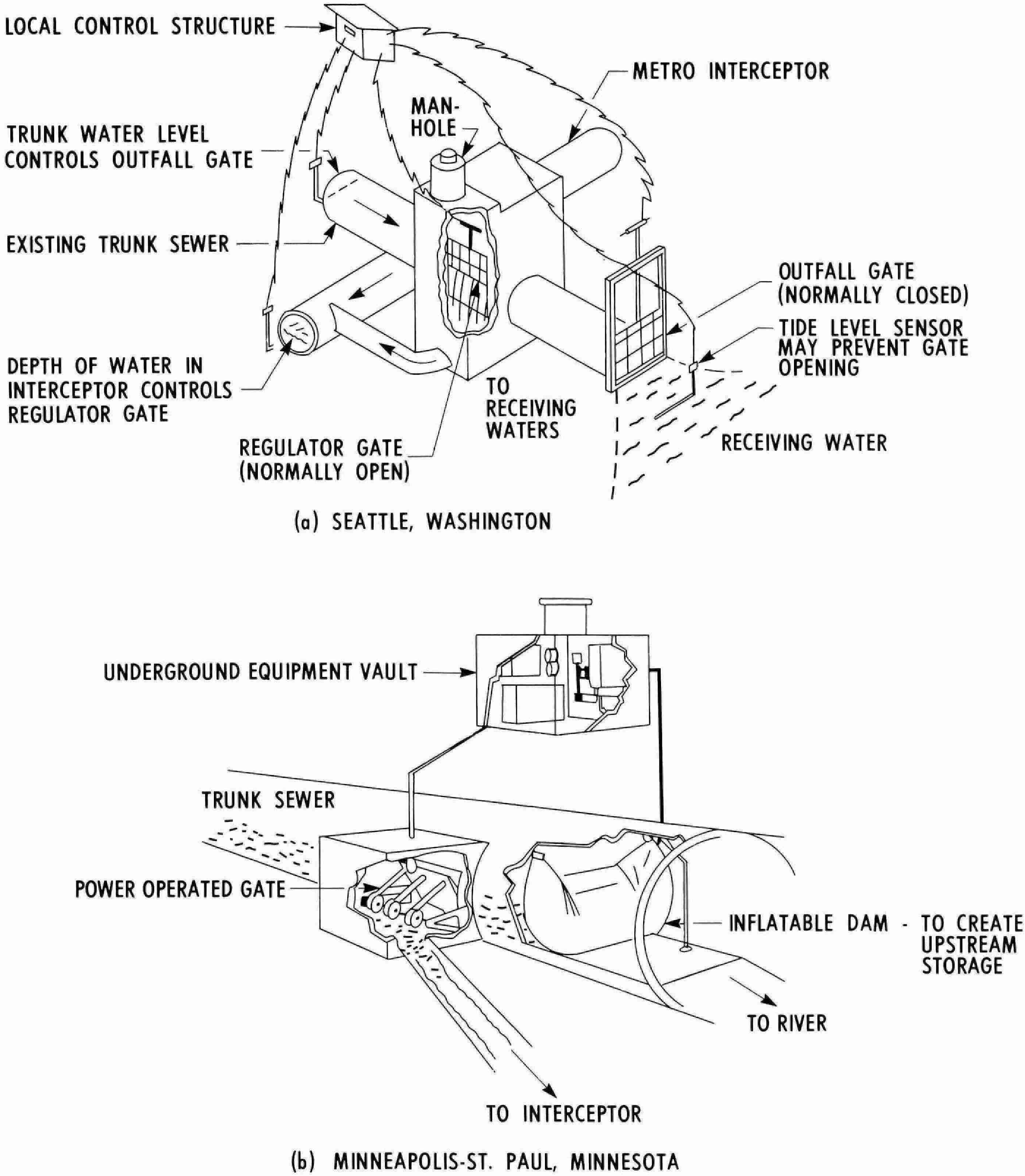


FIGURE 34. TYPICAL REGULATOR STATIONS FOR IN-LINE STORAGE SYSTEMS [5]

TABLE 21. INFLOW SOURCES AND CORRECTION MEASURES [2]

Type of Inflow and Originating Location	Possible Correction Methods
Usually in the Collection System	
Low-lying manholes	Manhole raising
Perforated manhole covers	Partial plugging or replace with watertight covers
Cross Connections (from separated sewers)	Plugging, diversion drainage
Back flow from receiver	Improved flap gate installations, changing overflow locations.
Drains from springs and swampy areas	Plugging, diversion drainage
Usually on Private Property	
Roof leader drains	Disconnection
Foundation drains	Disconnection
Yard drains	Plugging
Area drains	Plugging
Cooling water discharges	Disconnection

drainage. However, in many cases, some diversion of surface drainage may be possible.

Backflow from receivers has been noted as a significant problem in some sewer system studies in Ontario [6,7]. Backflow can cause dramatic loss of sewer capacity and/or flooding. Flap gates are a perennial source of operating problems and changing an overflow location or elevation can often only be done as part of major capital works projects (see Section 5.2.6.4).

5.2.6.2 Reducing infiltration. Table 22 [2] lists common sources of infiltration within collection systems, together with possible correction methods.

In combined sewers, dry-weather infiltration leads to loss in sewage treatment plant capacity and efficiency. The loss of dry-weather transmission capacity due to infiltration is not usually serious because combined sewers have much higher ultimate capacity. Further, wet-weather infiltration is usually overshadowed by the much greater influx of surface runoff during periods of precipitation. However, heavy infiltration in the period following precipitation or during snowmelt can potentially increase overflows, and hence may be of significance for wet-weather pollution control.

Remedial measures for infiltration reduction can be undertaken in conjunction with major sewer rehabilitation programs.

5.2.6.3 Comprehensive inflow-infiltration analysis. The manual "Handbook for Sewer Evaluation and Rehabilitation" [2] contains a comprehensive description of a recommended procedure for carrying out analysis of inflow-infiltration, currently in use in the U.S. The procedure is staged and geared to assessing the cost-effectiveness of removal of inflow-infiltration sources versus transmission and treatment. While inflow-infiltration surveys have been carried out in many Ontario cities, there is no experience in Ontario with the use of the whole highly systematized EPA procedure involving structured field interviews, flow monitoring, and rainfall simulation.

5.2.6.4 Improved regulator and flap gate operation. Regulators control the amount of combined sewage which is admitted to an interceptor sewer

TABLE 22. INFILTRATION SOURCES AND CORRECTION METHODS [2]

Sources	Possible Correction Methods
Collapsed pipe	Replacement; slip-lining
Broken or crushed pipe	Replacement; slip-lining
Cracked pipe	Slip-lining; lining with cement mortar or epoxy mortar; replacement
Deteriorated pipe joints	Chemical grouting
Leaking off-set joints	Slip-lining; chemical grouting; replacement
Open joints	Slip-lining; chemical grouting; replacement
Deteriorated mortar joints in brick pipes	Brick mortar replacement
Leaking house service connections	Chemical grouting; slip-lining; replacement
Faulty taps between sewers and manholes	Excavation and repair
Faulty taps between service connections and main sewers	Excavation and repair
Collapsed manhole and wet wells	Replacement
Deteriorated manhole walls, bases and troughs	Lining with cement mortar or epoxy mortar; chemical grouting; cement grouting
Deteriorated mortar joints in brick manholes	Brick mortar replacement
Deteriorated wet wells and pumping station structures	Lining with cement or epoxy mortar
Other sources such as deteriorated regulators, tide gates, etc.	Repair according to situation

rather than by-passed. Flap gates ("tide-gates") and similar devices are designed to prevent backflow of water from the receiver into the combined sewer system. When backflow occurs, sewer transport capacity is lost.

Effective operation of both types of control structure is therefore important in the management of a combined sewer system. Regulator station design should minimize unnecessary system overflow, while not exceeding maximum safe up-stream sewer levels.

The American Public Works Association (APWA) studied problems in combined sewer regulators and tide-gates [8]. They noted the need for improved installation, operation, and maintenance and produced a manual of recommended practice [9]. This manual also contains a description of the various types of regulators. Present day Ontario practice utilizes static regulator structures almost exclusively.

Automated regulator stations, suitable for operation and control in conjunction with remote telemetry are discussed in Section 5.2.5. Regulators which can provide treatment are discussed in Section 5.2.7.

5.2.6.5 Polymer injection. Injection of water soluble polymers into wastewaters has resulted in significant reductions in hydraulic friction, and consequently increases in sewer capacities. Increases in flow rate of up to 144% have been obtained under favourable conditions. Such flow rate increases are generally needed only for short durations, and polymer injections can boost the capacity of marginally inadequate sewers during critical periods. This will allow more storm flow to reach the treatment plant or assist in reducing upstream flooding due to surcharging. Three non-toxic polymers have been evaluated in full-scale applications: Polyox Coagulant-701, Polyox WSR-301, and Separon AP-30. Polymer concentrations in the order of 20 to 100 mg/L have been effective in causing flow increases up to 100% in a 46 cm diameter, 2440 m (18 in, 8000 ft) long sewer segment [5].

The polymer is normally added to an insoluble carrier, forming a slurry, and then fed at a rate proportional to the flow to achieve the desired polymer concentration. Polymer slurry is usually made by batch mixing of the components and stored under slight agitation until needed. Special polymer injection stations have been designed for portability. Application generally requires access to two manholes in the system; one

manhole is used for flow monitoring and the other for polymer injection. This is an expensive technique when judged from the viewpoint of operating cost alone. However, the capital costs of alternatives such as storage and pumpback, additional interceptor capacity, etc., may be much higher, making polymer addition more attractive on the basis of total cost.

5.2.6.6 De-separation of a separated sewer system. Overloaded sanitary systems, misused because they have many cross-connections to adjacent storm sewers, are the cause of basement flooding or sanitary overflow in some areas. When evaluating alternative methods of solving such problems, some consideration should be given to a systematic de-separation in areas with many random cross-connections.

De-separation involves the systematic interconnection of parallel storm and sanitary sewers to maximize the total available hydraulic capacity under peak flow conditions. Suitable off-line storage and routing controls must be added to the new "combined" system to achieve effective pollution control.

The circumstances and sewer system configuration in which de-separation may be feasible are likely very limited but the approach has been suggested in at least one case [10].

5.2.7 Measures to reduce pollutants in overflow

This group of control techniques can be used to achieve a reduction in the pollutant mass associated with a given volume of combined sewer overflow.

5.2.7.1 Catch basin elimination or replacement. Catch basins are intended to prevent clogging of sewers by rocks, sand, and other material washed from the roadways during storm events. During the early stages of a storm event, catch basins can be a source of first flush pollutant loadings from (separated) storm sewers or combined sewers. Various studies have assessed the loading contribution of catch basin sump accumulations. Results indicate that the liquid displaced from catch basin sumps during the early stages of storms may have a BOD_5 in the range of 50 to over 100 mg/L [11]. Further studies have indicated that in many cases the first flush problems caused by catch basins out-weigh any value they may have in reducing total pollution loadings.

It appears that proper design of catch basins could reduce pollution loads to the receiving water body. A recent EPA report [12] contains extensive discussion of catch basin technology and a recommended design configuration for new catch basins. To reduce pollutant loadings, more frequent cleaning would be needed to limit sediment accumulations to 40-50% of sump volume.

An alternative approach is to install catch basins only when the solids transport capacity of the downstream sewer is deficient, or at specific locations where sediment loadings to the sewer system are unusually high. Inlets cost considerably less than catch basins and savings would result. In those new developments which include a terminal detention-sedimentation pond, savings from not installing catch basins could be off-set against the cost of the ponds.

Methods to reduce first flush pollution from existing catch basins include:

- more frequent sump cleaning to ensure adequate pollution control function (frequency would depend on local conditions);
- elimination of sumps in some areas;
- installation of alternate sediment-retaining devices within suitably modified catch basins. This alternative requires further development, but one approach involves use of a mounted reusable bag positioned below the inlet grating. The bag can be cleaned easily and does not retain a reservoir of polluted water. This approach might eliminate the first flush problem without sacrificing solids removal capability. The bag system has been used in Europe [13].

In new construction, sewers are generally designed for self-scouring velocities. For this reason, it has been suggested that even where high sediment loads are expected, catch basins be eliminated and flows allowed to pass through the storm water system to primary sediment collection points which can be more readily serviced. There is no proven means to achieve this at present. Modification or adaptation of the Swirl/Helical Bend separator, or use of other separators would be necessary, supplemented by devices for obtaining a dry 'grit' fraction.

5.2.7.2 Sewer flushing. The rationale for combined sewer flushing arises from the accumulation of sediment during low velocity (dry weather) flows, especially in those sewers with low slopes. Under storm conditions, the deposits are flushed out to the treatment plant or to overflow. Sewer deposits have been cited as largely responsible for first flush conditions during the early stages of storm events, resulting in discharges of high levels of SS and BOD_5 [2].

Extensive flushing studies were carried out by the FMC Corporation under contract to the EPA on 30 and 45 cm diameter (12 and 18 inch) combined sewer laterals. The study included the development of a portable prototype flush station using settled sewage as the flushing liquid [14].

Regular sewer flushing of a critical section of trunk sewer in Boston reduced BOD_5 and SS in overflows by 50 and 20 percent respectively [15]. Other findings indicated that about 10.3% of the daily input of solids settled in the sewers and that 50% of all accumulations were contained in 100 critical sections. Study recommendations included initiation of a large-scale flushing program, with introduction of the flush volume by water tanker rather than use of automated flushing systems.

Technology relating to the deposition of solids in sewers and their systematic flushing is still under development. Recent developments have included publication of detailed procedures for estimating dry weather pollutant deposition in sewerage systems [16].

5.2.7.3 Regulators which provide treatment. Regulators exist which can achieve enhanced suspended solids removal from overflows simultaneously with their regulating or "flow-splitting" function. That portion of the flow routed onwards to the interceptor is "enriched" in suspended solids.

Detailed design methodology has been published for two devices: the Swirl Concentrator, and the Helical Bend Separator [17,18]. The Swirl Concentrator has been demonstrated at full-scale and a general description of the device is given in the Section 5.4.3.

The Helical Bend Regulator is an expanded curve section of a combined sewer (Figure 35). The device utilizes "the secondary flow patterns produced by helical hydraulic phenomena to deposit solids" [17]

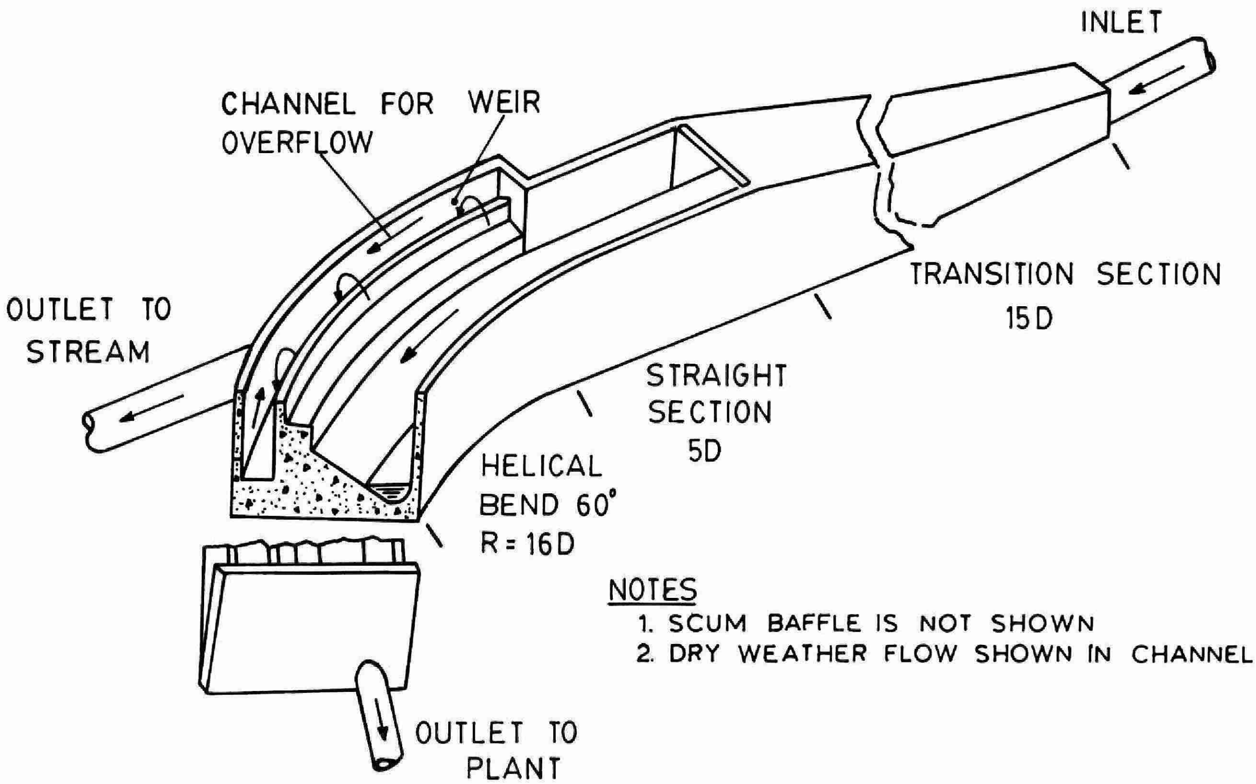


FIGURE 35. ISOMETRIC VIEW OF HELICAL BEND REGULATOR [18]

at the inside bottom part of the curve. Floatable materials are prevented from overflowing by a baffle in front of the side spill overflow weir (the baffle is not shown in Figure 35). The basic device was developed in the U.K., where six to seven times the dry weather flow (DWF) is commonly intercepted. Extensive hydraulic and mathematical investigations were carried out by the APWA [17] on a modified geometry more suited for an interception rate of three times DWF, and Figure 35 shows the final design arrangement of the APWA device. A mechanical gate can be installed on the outlet to the water pollution control plant (WPCP) to dynamically vary the interception rate.

The APWA study also compared the Helical Bend Regulator with the Swirl Concentrator. A Helical Bend Regulator (1.4 m^3/s (50 cfs) design capacity) requires about 2340 m^2 (25 200 ft^2) of area, whereas a Swirl Concentrator of like capacity occupies only 1440 m^2 (15 500 ft^2) and has a lower capital cost.

No full-scale operating results are yet available for this device. However, a three-year study of the U.K. prototype has recently been concluded [19].

5.2.8 Major structural changes to a combined sewer system

Other than separation, the possible physical modifications (structural changes) to a combined sewer system are additional or larger conduits and storage facilities to transport or hold the wastewater, and/or the addition of automated regulators and supervisory control facilities to enable effective routing of wastewater and management of overflows. The latter options have been addressed in Section 5.2.5. The addition of conduits and storage is discussed briefly in the following sections. Various aspects of storage and storage/sedimentation are discussed further in Sections 5.3 and 5.4 of this chapter.

5.2.8.1 Increased interceptor capacity. For economic reasons, interceptor design capacity is usually in the range of 1.5 to 5 times DWF with the majority of interceptors in North America being in the range of 2-3 times DWF. Typically, the latter range yields an overall mean annual sewage collection efficiency of between 96% and 98% [20]. Increasing the interceptor capacity further results in only small increases in collection efficiency, at best, and is quite costly as treatment plant capacity must also be increased to match increased conveyance. Other approaches to providing adequate collection efficiency to minimize overflows will often prove more cost-effective than enlarging the interceptor sewer. As well, enlarging interceptor capacity may be more disruptive of surface activities, such as traffic movement, than other alternatives.

5.2.8.2 Upstream storage with controlled release rate to trunk sewers. Upstream storage is loosely defined here as open or enclosed storage not far downstream of the point at which runoff enters the collection system. It can include local detention of all or a selected part of the runoff in underground silos, large diameter sewers and the like. Many storage points may be needed to reduce overflows by this means.

The concept of local storage of combined sewage is unlikely to find wide acceptance. However, it is also possible to preferentially retard road runoff by local storage to reduce the peak flow in the combined sewer. Like other upstream storage techniques, this may help reduce the cost of downstream components of the collector system (see Section 5.3).

5.2.8.3 Regulator consolidation and off-line storage. The number of regulator structures within even a small combined sewer system can be much too large to consider automated control of all regulators or storage at each overflow location. Regulator consolidation, in which scores of overflow points may be reduced to a dozen or less, is a necessary first step toward a more manageable system.

Initially, hyraulic analysis of the sewer system is carried out to establish the conditions under which overflow occurs at various points. Analysis is then extended to include the effect of off-line storage on overflows at different points in the system. The number of overflow points is consolidated to the minimum number necessary for safe, effective system management, and those remaining are automated. Figures 36 A and B, illustrate the type of analysis which can be performed to examine the effectiveness of consolidation.

Regulator consolidation is a logical extension of routing control in larger combined sewer systems. In smaller systems which do not have significant sewer storage volume, off-line storage facilities can be designed to permit the subsequent return of contents to the interceptor sewer, or constructed along with satellite wet weather treatment facilities. In either case, the design procedure should optimize the cost of the storage-treatment combination.

5.2.8.4 Deep tunnels and downstream storage. This option represents a major structural alternative to sewer separation and has been adopted in principle by a number of large cities in the U.S. as a major component in their master drainage plans.

One advantage of deep tunnels is that they can be installed in such a manner as to minimize disruption of surface activity and other sewerage during construction. Overflows are collected or eliminated by routing the combined wastewater through drop-shafts into specially constructed deep tunnels. The tunnels provide considerable storage in themselves but also transport the combined wastewater to off-line downstream storage for later treatment.

Tunnels offer the possibility of achieving a high degree of control over the flow, routing, and treatment of combined wastewater. Also the point of discharge of overflows from storms exceeding the design capacity can be controlled.

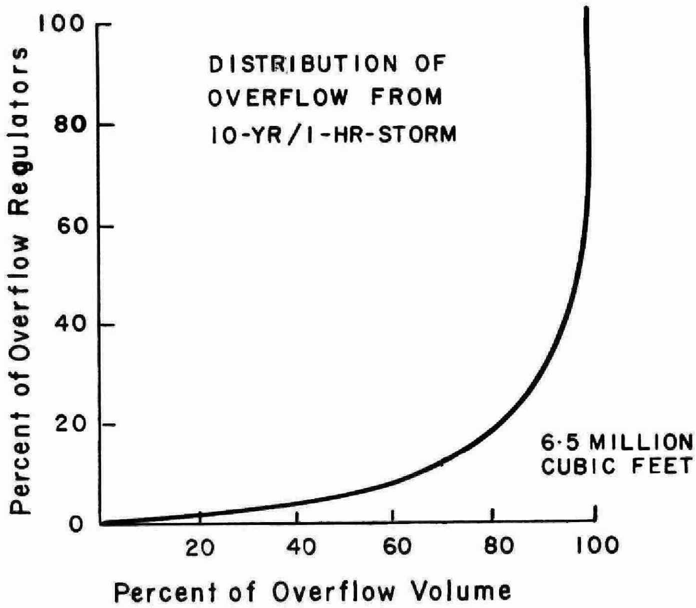


FIGURE 36A. DISTRIBUTION OF OVERFLOW VOLUME AMONG REGULATOR STATIONS BEFORE CONSOLIDATION

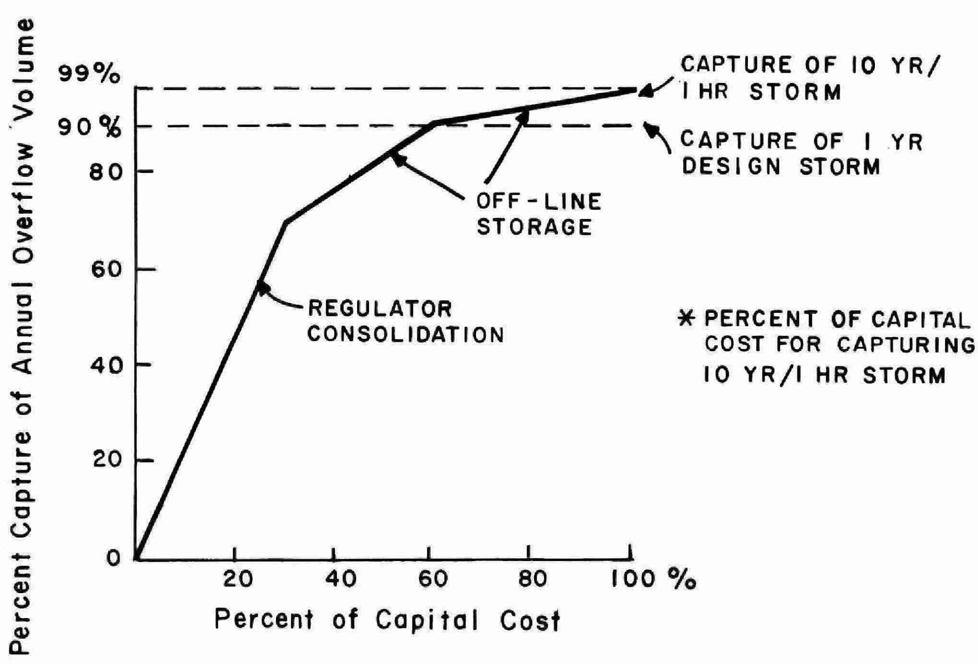


FIGURE 36B. EVALUATING THE COST OF REGULATOR CONSOLIDATION PLUS OFF-LINE STORAGE

The resulting systems are complex and pumping may be necessary to elevate wastewater from the tunnels. They certainly require telemetry, central control, a high degree of active system management, and good operation to be successful. No city-wide schemes have been implemented to date in the U.S. or elsewhere. This type of system is probably not applicable to small or medium-sized cities.

5.2.9 Separation in combined sewer systems

Sewer separation is the traditional method of alleviating flooding and pollution due to combined sewer overflows. Until comparatively recently, it was the only method available.

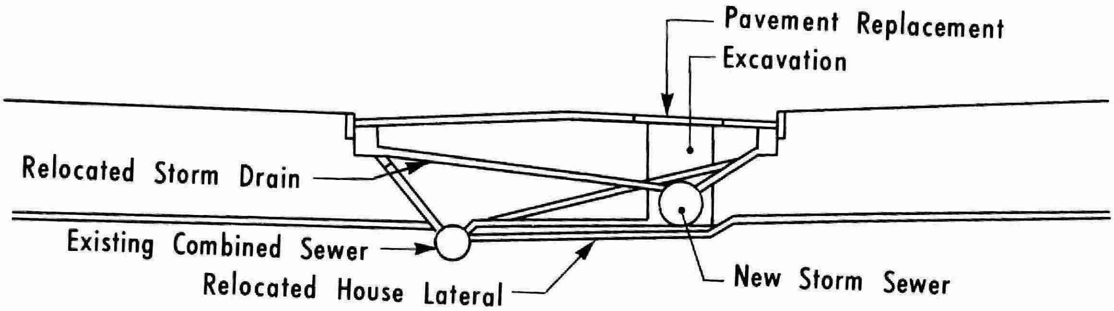
The following sections summarizing the various methods of separation are based on a report by Marsalek [20].

5.2.9.1 Conventional separation. Conventional methods of sewer separation are pictured in Figure 37. Sewer separation can be accomplished in two ways: under Scheme A (Figure 37A), the existing combined sewers are used only as sanitary sewers and new storm sewers are constructed. Under Scheme B (Figure 37B), the existing sewers can be used as storm sewers and new sewers constructed to carry sanitary sewage. The new conduit carrying sanitary sewage (of smaller diameter than the storm sewer) is usually placed a few feet below the existing combined sewer so that the house or other laterals can be connected.

Under either scheme, roof leaders are disconnected and diverted onto the ground to drain away from the house foundations onto grassed areas. Often, this is not practicable, due to small lot sizes, adverse grades or soil conditions, and in such cases roof leaders are usually directed down driveways to street catch basins. The redirection of weeping tile flows away from the sanitary sewers involves significant disruption and expenditure on private property for which cost allocation is problematic. It is rarely carried out concurrently with separation, although zoning regulations may require that separate weeping tile service be installed, site by site, as property is redeveloped. The end result of current practices is that conventional separation is rarely carried to completion in a rigorous sense.

Since separation is usually carried out for the relief of basement flooding and/or to permit future development, Scheme A is most often used.

SCHEME A



SCHEME B

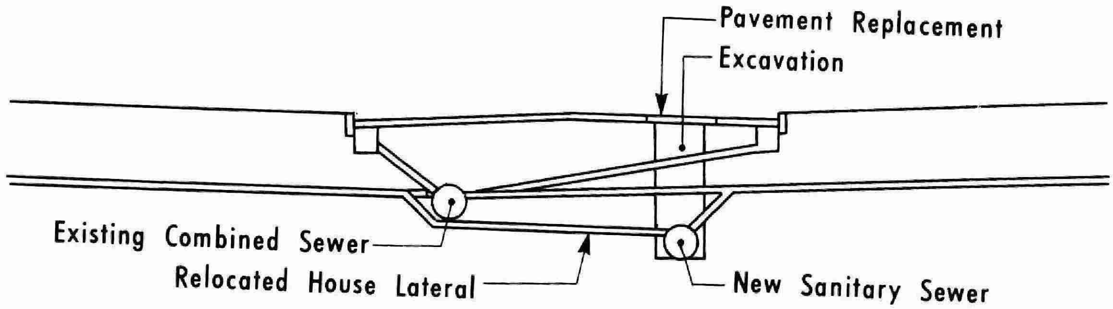


FIGURE 37. TWO SEWER SEPARATION SCHEMES

Planning of sewer separation schemes, whether over a short or long-term time scale, should include a detailed evaluation of the environmental effects during various phases of implementation. The purpose of the evaluation would be to ensure that environmental benefits are gained simultaneously with flood relief, and that the duration or volume of overflow is not increased during implementation through poor project planning.

The ultimate pollution loadings associated with the (separated) storm water should be considered and offset against reductions in combined sewer overflows. Sewer separation alone may not be effective in reducing storm water pollution to the level needed to achieve the required water quality in the receiver.

5.2.9.2 Separation of road drainage. Previously described separation methods dealt with "complete" sewer separation. Partial separation of combined systems can be carried out by building a new system of road sewers to carry a significant portion of the surface runoff from roads, parking lots, and sidewalks, which usually comprise 50% of the impervious areas. Storm water from roofs continues to enter the existing combined sewers. Partial separation costs about half as much as total separation, which makes it an attractive option for flood relief. The following pollution control factors should be considered when evaluating this alternative:

- Storm drainage from roads may be sufficiently polluted that separate abatement measures will be needed.
- Large volumes of storm flow will still enter the sanitary system, which will continue to respond like a combined system in wet weather. Offline storage or other pollution abatement methods may be needed.
- Relief from the sanitary to the storm sewers is undesirable since it merely changes the effective point of overflow.

5.2.9.3 Express sewers. 'Express' sewers represent another form of partial sewer separation. These convey sanitary flows from new developments with separated sewers, or from newly separated areas, directly to the treatment plant, avoiding or passing straight through areas served by

combined sewers. Hence, hydraulic flows in downstream combined sewers are maintained at existing levels or reduced, minimizing the overflow volumes. Flows from the separated portion of the collection system are given 'priority' at the treatment plant.

The cost-effectiveness of this approach requires individual evaluation in each case.

5.2.10 Applications of collection system controls

Combined Sewer Flushing - Boston, Massachusetts [21]. A high-density residential area of about 1460 hectares (3600 acres) in Boston, Massachusetts, served by combined sewers, was studied to determine whether rehabilitation of the aging sewer system was a viable alternative to providing new facilities.

The program involved preparation of comprehensive sewer maps depicting the hydraulic characteristics of the interceptors, trunks, and collection system. Over 168 km (550 000 ft) of the collection system were catalogued for hydraulic properties and for predictions of solids deposition. An empirical model was used to predict dry weather flow deposition in a single length of pipe. Variations of this model were developed to predict amounts of daily deposition in the collection system network. A total of 3000 segments with average lengths of 55 m (180 ft) were needed to represent the study area. A master schedule was prepared for the 30 subcatchments comprising the system, showing the locations with moderate to heavy deposition. Sediment accumulation rates and priorities for flushing were established. The significant findings were that 10.3% of the daily input solids would deposit, with 50% of total accumulations depositing in 100 sections of pipe, and 75% of accumulations in 420 segments.

An empirical model was developed for predicting theoretical flushing volume requirements for each of the 420 segments showing significant dry weather deposition.

Experimental sewer flushing indicated that introduction of the flush volume by water tanker would be superior to a fully automated flushing system.

To improve the existing system, consideration would be given to the removal of sediments which presently reduced the nominal interceptor

capacity by about 50%, together with a flushing program covering critical sections.

Costs of dry weather flushing alternatives for Dorchester Bay (Boston) were estimated as follows:

Number of Sewer Sections*	Capital** ($Million)	Annual O & M
40 pipe sections	0.88	0.022
240 pipe sections	2.28	0.132
487 pipe sections	4.02	0.268
867 pipe sections	6.67	0.477
1567 pipe sections	11.57	0.861

* Average section length is 55 m (180 ft).
**Capital cost based on ENR concentration costs index 2200.

<u>De-separation - Lakewood, Ohio [10]</u>. Sewer de-separation has been proposed at Lakewood, Ohio. A hydraulic study on the existing, badly infiltrated, separated sewer system was initiated because of frequent complaints of basement flooding throughout the serviced area and known overflows to Lake Erie during most storms. The total catchment area was 1250 ha (3100 acres) of mostly residential land. About 1050 ha (2100 acres) are served by a separated sewer system with sanitary and storm sewers laid in a common trench, and about 200 ha (500 acres) are served by combined sewers. Of the 266 km (875 000 ft) total length of sewers, 120 km (393 000 ft) are sanitary, 106 km (348 000 ft) are storm, and 40 km (133 500 ft) are combined sewers. The system also includes 33 regulators.

Computer simulation studies confirmed that the sanitary sewers were drastically undersized to handle the storm water entering them at the many points of storm sewer cross-connection. The analysis also indicated that the system could be adapted to operate as a combined system, using storm and sanitary sewers in parallel. This could be accomplished by providing adequate crossover capacity, replacing some inadequate sanitary sewers with adequate combined sewers, and installing control regulators to minimize overflow. To eliminate all basement flooding, about 35% and 47%

of the combined and sanitary sewers, respectively, should be replaced or relieved. Cost of this project was estimated at about one-third the cost of replacing the aging sewer system. The sewer system in this area was particularly suitable for de-separation because the separated and combined systems shared common trenches and common manholes with easy interconnection possibilities at each manhole.

Sewerage improvement is planned in three priority phases. Work under groups one and two will eliminate practically all basement flooding, while group three improvements will remove system bottlenecks and rehabilitate deficient sewers. A cost summary follows:

Priority Group	New Sewer Length (m)	(ft)	Control Structures	Complaints Eliminated	Cost
1	11 600	38 065	8	82%	$ 5 683 300
2	22 590	74 110	2	15%	8 488 060
3	25 305	83 025	0	-	10 148 825
Total	59 495	195 200	10	97%	24 320 185

This alternative was much less costly than others proposed for flood relief. The modified system would be suitable for operation with off-line storage to reduce pollution due to overflows.

Polymer Injection [5,22]. Polymer injection was tested at Richardson, Texas [5] using polymer concentrations of between 35 and 100 mg/L, which decreased flow resistance sufficiently to eliminate surcharges of more than 2 m (6 ft). The capital cost of the portable, non-automated installation serving a 60-cm (24-inch) diameter, 1250 m (4100 ft) long sewer was $8200. Polymer cost, based on using Polyox WSR-301V, was estimated at $0.17-0.46/m^3 of sewage ($0.63-1.76/1000 U.S. gallons).

At Dallas, Texas [5] a portable prototype polymer injection station was installed for relief of surcharge-caused overflows at 15 points along a 2440 m (8000 ft) section of 45 cm (18 in) diameter sewer. During storm events, the infiltration ratio approaches 8 to 1. The

station is a 3 m (10 ft) diameter by 8 m (26 ft) high sheet metal covered building. The upper half provides storage for 6350 kg (14 000 lb) of dry polymer and contains dehumidification equipment. The lower half contains a polymer transfer blower, a polymer hopper and agitator for dry feeding, a volumetric feeder eductor, and appurtenances. The unit is self-contained requiring only power and motor hookup. The unit is also set up for fully automatic operation and may be started by external level sensors located upstream and downstream from the injection site. The polymer is expected to reduce the surcharge by about 6 m (20 ft) over the first 1220 m (4000 ft). Feed rate is expected in the order of 2.3 kg/min (5 lb/min) but will be flow-paced to give a polymer concentration in the sewer of about 150 mg/L. It is expected that the unit will be in operation about five times per year and that surcharge reduction will be complete about seven minutes after the start of polymer addition. Construction cost for the unit, including installation at the site, was $146 000 (ENR 2000).

Based on a polymer concentration of 150 mg/L (Polyox WSR-301V), polymer cost would be $0.70 per 1000 L of sewage ($2.64/1000 U.S. gallons).

The above experiences occurred in gravity flow situations. In a more recent application, polymers have been used in Bergen County, New Jersey [22] to increase the flows from a pumping station through a downstream force-main. Using an anionic polymer (Hercufloc 831), a 29% increase in flow rate was achieved at concentrations of 100 mg/L. The application was regarded as cost-effective in eliminating upstream by-passing, which had occurred once or twice a year.

Flow Routing in Combined Sewers at Seattle, Washington [5]. This flow-routing system was first operational in late 1971, and now has 10 fully-equipped regulator stations, such as the one shown on Figure 34, with three more under design. All stations are monitored, and are designed so that they may be operated by a supervisor from a central control console. The estimated maximum safe storage in the trunk lines and interceptors is 121 000 m^3 (32 million U.S. gallons), roughly equivalent to 13 mm (0.05 in) of direct runoff from the combined and partially separated sewer areas. Interceptor capacity is generally three times the estimated year 2000 dry weather flow. Under supervisory control, overflow volume has been reduced by approximately 52 percent. Fully-automated control (CATAD)

was attempted in 1973. CATAD is a computer-controlled total system management concept, taking advantage of in-line storage to limit overflows, and with the capability to select overflow points based on receiving water quality data. The system presently includes 15 fully-equipped regulator stations and one major pumping station. When operated under limited automated control for a three month period the overflow volume decreased by more than 90%, and pollutant loadings were found to be 68% lower.

When fully predictive capability to program decisions is added, it is expected that overall reductions in overflow of 80% will be consistently achieved.

5.3 Storage

Storage involves the use of various techniques to accommodate wastewater or surface runoff between entry to the collection system and discharge to a receiver.

Figure 38 illustrates the many potential storage locations within a drainage system. On-site controls have been discussed in Chapter 4. The purpose of this section is to outline the principal considerations in the design and use of storage for single and multi-purpose use. Pollution control considerations and design criteria for storage/sedimentation facilities, while considered briefly in this section, are discussed in greater detail in Section 5.4.

5.3.1 Types of Storage Facilities

Based on location in the drainage system, storage facilities may be designed as in-line or off-line, at upstream or downstream portions of the collection system. Based on function, storage facilities may be designed for single or multi-purpose use, for detention or retention. Typical storage systems are represented schematically in Figure 39. Storage facilities may be open or enclosed, as reservoirs or covered tanks. Reservoirs may be natural or man-made surface depressions (Figures 40, 41).

5.3.1.1 <u>In-Line (on-stream) storage</u>. The surface runoff and wastewater generated in urban areas is most often collected in subterranean pipe networks. As most storm water runoff enters the sewer system by way of street gutters and catch basins, storage in catch basins, large volume

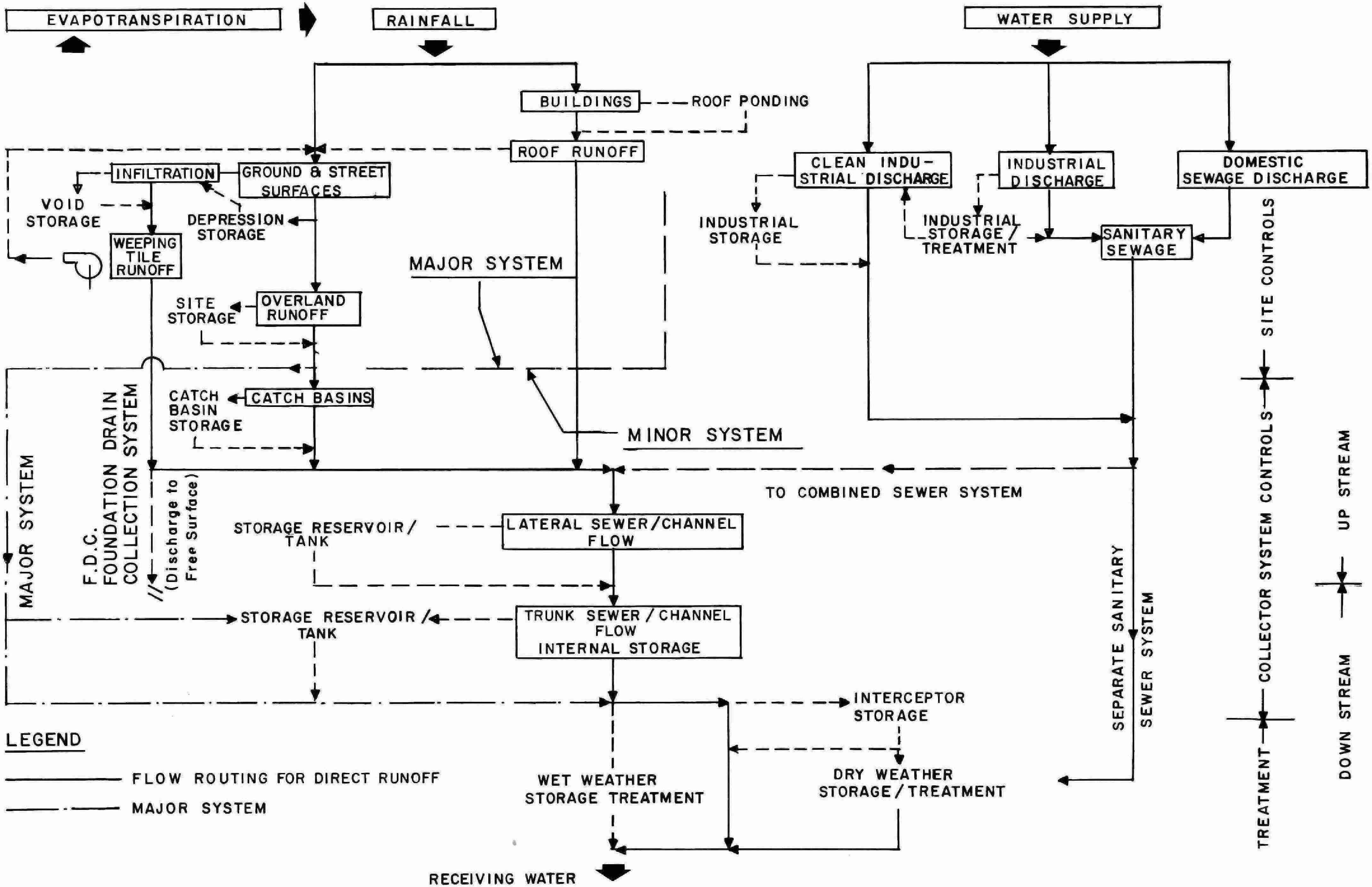


FIGURE 38. ALTERNATIVE STORAGE LOCATIONS IN COMBINED AND SEPARATED STORM SEWER SYSTEMS

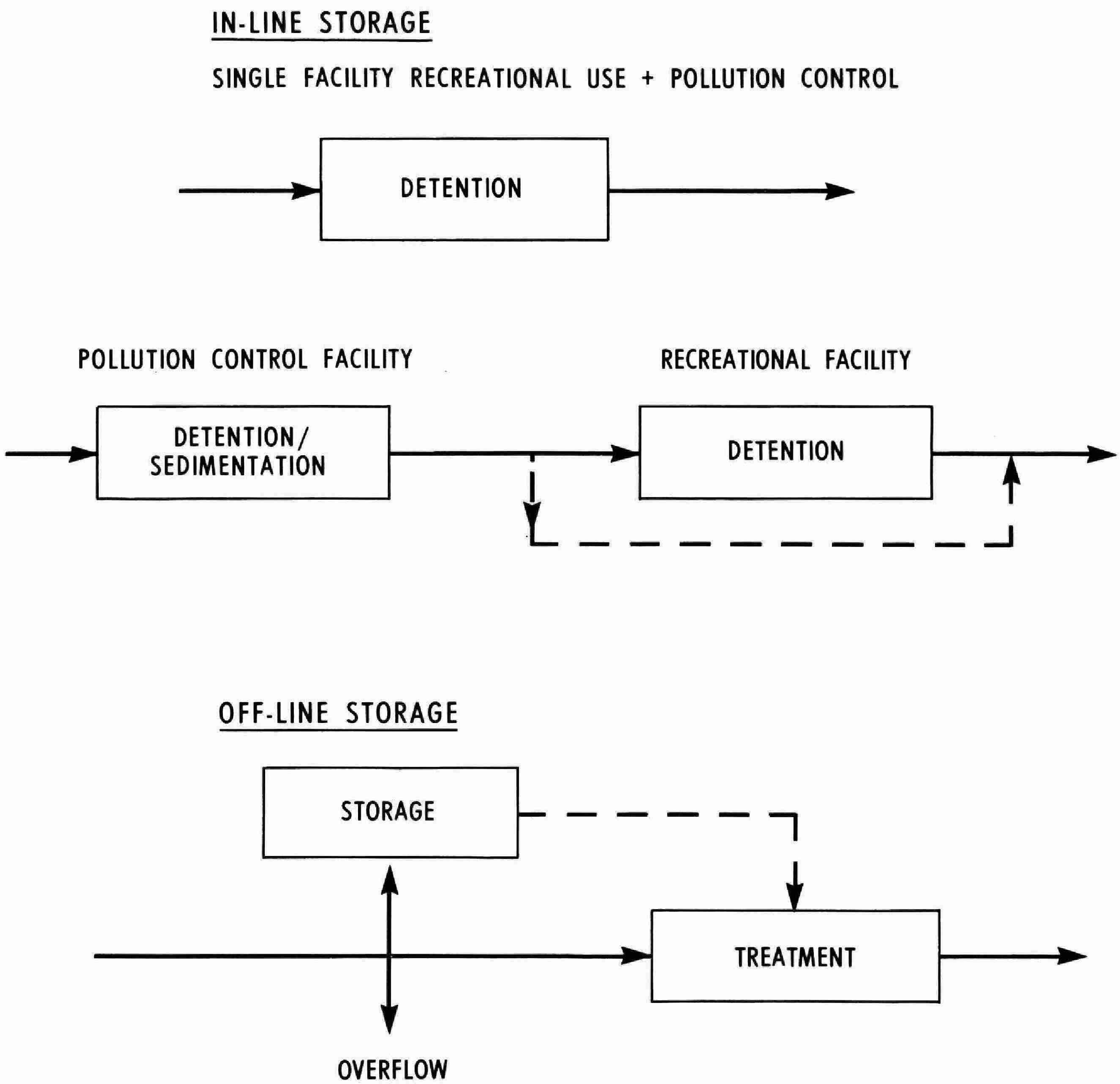


FIGURE 39. SCHEMATICS OF TYPICAL STORAGE SYSTEMS

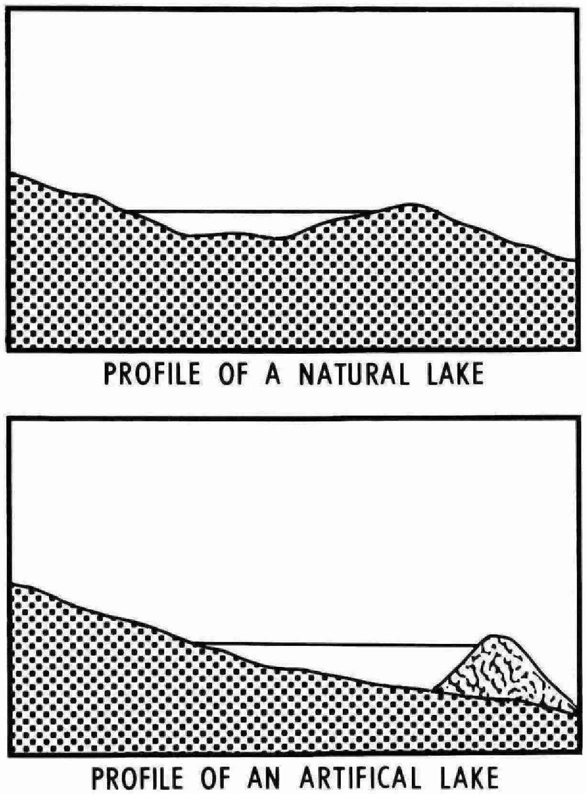


FIGURE 40. TYPICAL PROFILES OF A NATURAL AND ARTIFICIAL LAKE [23]

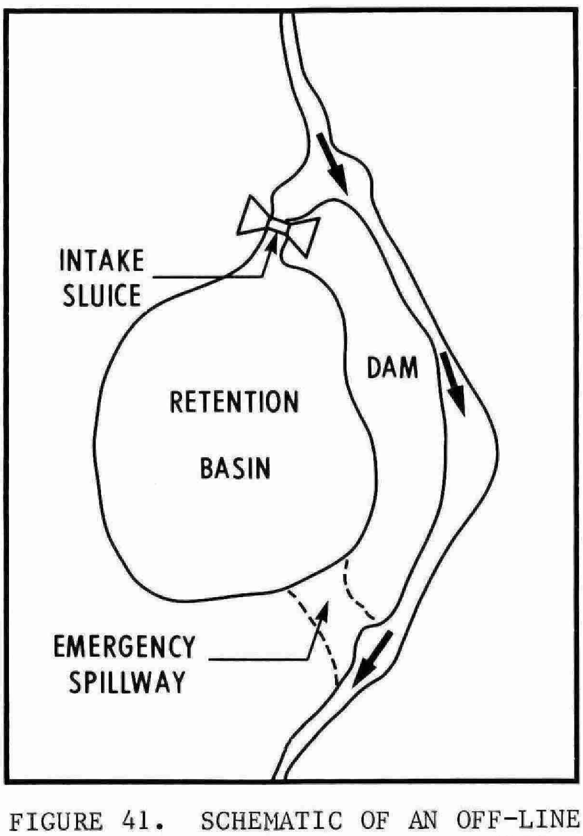


FIGURE 41. SCHEMATIC OF AN OFF-LINE STORAGE RESERVOIR [23]

inlet manholes, or street silos can potentially be used to detain street runoff before its release to the storm or combined sewer. This technique is particularly useful for the relief of local basement flooding. To control the outlet flow rate from storage a flow-regulating device of known and dependable characteristics should be used.

Urban drainage collection systems are designed to convey wastewater from surface runoff at a definite peak flow rate, usually that resulting from a two to ten-year design storm. Hence, for less significant storms there is an unused excess volume within the conduits. The excess volume may be utilized for storage by providing control structures to restrict the flow which then backs up into upstream sewers. A control mechanism must automatically remove or by-pass the restriction to flow when critical levels are reached. In-line storage has been used in the larger conduits of combined sewers to reduce total overflow volume, and for control of the peak rate of discharge in both storm and combined sewers. Flat sewer grades are essential to accumulate worthwhile storage volume when surcharge must be avoided.

Parkland and greenbelt areas, where suitably located in relation to open-channel drainage ways, can provide areas where storm runoff can spread and be stored for slower delivery downstream. Additional major storage facilities, such as ponds, basins or tanks, may be required to supplement the storage which is available in channels and sewers.

5.2.1.2 Off-line (off-stream) storage. Off-line storage devices include ponds (Figure 41), primary settling tanks, underground silos, underwater bags, void space storage, and deep tunnels. Within a large drainage system, off-line storage can also be achieved by directing wastewater or surface runoff away from full and surcharged conduits experiencing high flow rates to those less full, with lower flows. This is possible because of the movement and uneven distribution of precipitation over a large drainage area. This technique is very effective in the case of combined sewer systems where individual drainage networks are interconnected through interceptors, i.e., in large systems storage can be made available both in "space" and in "time".

5.3.1.3 Upstream/downstream. Upstream storage (Figure 38) can be used to achieve passage of runoff through the sewer system in a selected sequence. Additionally, it can lower the cost of downstream elements of the collector system and treatment facilities.

Downstream storage can be used to protect the receiving water from excessive peak flow rates or pollutant loadings, and can provide flow equalization to a treatment facility.

5.3.2 Design considerations

The following should be considered when designing single and multi-purpose storage facilities:

- defining runoff quantity,
- defining runoff quality,
- physical design parameters,
- inflow/outflow controls,
- operation/maintenance procedures,
- secondary uses (and aesthetic considerations), and
- safety and health aspects.

5.3.2.1 Defining runoff quantity. Design of storage facilities requires a method for computing the time distribution of runoff quantity for real storm events or design storms. The use of a hydrograph method is necessary in order to account for the variations in flow rate with time and their effect on the volume stored. The required storage volume depends on the form of the inlet hydrograph and proposed outlet hydrograph (see Chapter 3). Within a watershed, careful attention must be paid to the design of major storm water storage facilities. If these are not properly planned, the overlap of runoff rates may produce higher peak flows than would otherwise have occurred. The water quantity design aspects of reservoirs to be built in Ontario are subject to review and approval by the Conservation Authority Branch of the Ministry of Natural Resources.

5.3.2.2 Defining runoff quality. A calculative method may be required for assessing the effect of storage upon quality when:

- Storage is being used to simultaneously obtain treatment. The quality of both inflow and outflow are then required to obtain the treatment efficiency.

- The discharge from storage goes directly to a receiver. Knowledge of quality may be required to assess impact.

In routing flows through storage, the effects of mixing regime upon quality should be considered. Initially, solids deposition, pollutant decay, etc., can be calculated based on the assumption of either complete mixing or plug flow. The initial assumptions can be modified by applying appropriate efficiency factors at final design.

5.3.2.3 Physical design parameters. The major physical design parameters (depth, surface area, shape and slope of embankments) will vary widely depending on the type of structure, the availability of space, and other factors. These factors are discussed in detail in several recent publications [5,23,24,25,33].

5.3.2.4 Inflow controls. Inflow controls for off-line facilities include interceptor structures, sluices, valves or gates, and pumps. In the case of surface runoff, protection of the main facility from sediment accumulation may be provided by an upstream siltation basin (see Section 5.2.5).

5.3.2.5 Outflow controls. There are two basic outflow control alternatives depending on the type of runoff and the economical level of control. For control of CSO, the objective is to capture and retain the runoff for subsequent treatment. Therefore, outflow structures (commonly weirs) are not required to function until storage is full or nearly full and must then release all or nearly all additional runoff. System design must prevent flooding throughout upstream drainage elements. In the case of storm water runoff, storage is generally provided to reduce peak flow rates, i.e., to release flow in a controlled manner from a storage facility. A percolated pipe spillway, orifice, compound weir or pump may be suitable for this purpose. In all cases provision of emergency spillways, diversion channels or the like is desirable, to protect the storage facility against over-topping or damage from extreme storm events.

5.3.2.6 Operation and maintenance. Both in-line and off-line combined sewer storage facilities require active operation and maintenance. Further, they may be located underground, or require covering which necessitates controlled ventilation. In-line facilities require level indicators, and/

or other instrumentation to monitor flow and quality. Flushing systems or mechanical equipment are utilized for solids removal in storage-sedimentation tanks. Removal of solids deposited during in-sewer or deep tunnel storage, is reported as a definite problem [25]. Mechanical mixing may also be necessary if sedimentation is undesirable during storage [5]. Off-line facilities are generally intended to provide storage for treatment processes. These facilities are usually large, and of more sophisticated design than in-line storage facilities. Since flow by gravity into and out of storage is not always possible, off-line facilities may include pumping equipment, associated valves, and piping. Details of maintenance problems in storage facilities for combined sewage are given in several recent publications [5,24,25,26,33].

Storm water storage facilities, especially in-line facilities, are simpler than those for the storage of combined sewage, and hence usually require a much lower level of active operation and maintenance. These facilities are generally located at ground level as reservoirs. Multi-purpose use of these facilities is much more easily achieved than for combined sewage storage facilities.

5.3.2.7 Secondary use, safety, and health aspects. Permanent ponds and lakes can provide multiple benefits including enhancement of property values and urban landscape. Potential exists for recreational boating, ice skating, fishing, and habitat for resident or migratory wildlife [27]. Protection from health and safety hazards and positive measures for control of aesthetics, including odour control and visual appearance, should be integral parts of storage planning and design. The structural integrity of permanently filled reservoirs and their retaining dams is also a factor which must be carefully considered during design. In Ontario, the Ministry of Natural Resources has approval authority over dams through the Lakes and Rivers Improvement Act. The safety of members of the public using facilities for recreation should be carefully considered; the control over factors such as maximum depth of water, near-shore slopes, etc., is generally within municipal jurisdiction.

Excessive lowering of the water surface by evaporation or infiltration can decrease the aesthetic and recreational value of storage. The ability of permanent storage sites to retain water should be carefully

evaluated and the possible need to supplement the water supply from wells or other sources should be investigated. Siltation of permanent ponds may result in loss of storage capacity or undesirable weed growth [27], and eutrophication or declining water quality can be a serious problem in shallow lakes [28]. Ongoing or periodic maintenance programs and effective enforcement of sewer use by-laws are required to prevent these undesirable conditions.

Since the water stored in "dry ponds" is expected to drain completely after a storm, the dry ponds can potentially serve dual purposes. Golf courses, recreation fields, and parkland are examples of potentially compatible uses [27].

5.3.3 Cost

Sizing of storage is usually determined on the basis of cost-effectiveness, i.e., the increased cost of storage is offset against some saving or other benefit. Economic trade-offs are site-specific and a large number of variables are involved, so any generalized estimating method for storage costs is subject to many uncertainties. The following sections briefly discuss three major components of total cost - land, construction, and maintenance costs.

5.3.3.1 Land costs. Land costs are by far the largest expenditures for open-space detention ponds and basins. In contrast, underground storage may require little or no permanent dedication of land, and in-sewer storage has no land requirement at all. In the case of a storm water detention facility, land costs can be offset by utilization of parks or greenbelts. The increase of real estate values around the shoreline of a well-planned, aesthetically-pleasing facility, can sometimes more than offset land costs.

5.3.3.2 Construction costs. Underground facilities require major expenditures for excavation, structures, and accessways. Construction costs can also be significantly increased if sophisticated equipment for control, telemetry, and services is needed. Reservoirs usually have much lower construction costs because conventional earth-moving equipment can be used for excavation and the construction of berms and dams. In locations where permanent storage ponds are required for runoff control,

temporary ponds used in erosion-sediment control programs at construction sites can often be planned in such a manner that they can be altered to become permanent at low additional cost.

Tables 23 and 24 show U.S. cost data for several types of detention facilities as presented in a recent APWA report [29]. Some recent Canadian cost data are presented in Table 25. The EPA publication "Cost Estimating Manual - Combined Sewer Overflow Storage and Treatment" [26] provides additional cost data for storage facilities. The United States Soil Conservation Service Publication, "Storm Water Management Cost Study" [30] discusses various aspects of land and construction costs for storm water management ponds, and provides an outline of methodology for predicting the total costs of storm water management for a catchment using either one large detention basin or many smaller basins.

5.3.3.3 Operating and maintenance costs. Operating and maintenance costs for storage facilities are usually grouped together and depend largely on the system complexity and whether simultaneous treatment is being provided. For combined sewage detention facilities, a recent EPA manual [26] provides comprehensive information.

The annual base maintenance cost for lakes is generally about \$370 per hectare of lake surface per year (\$150/acre/year) - excluding silt removal. Maintenance of grassed areas around impoundments can cost \$990/ha (\$400/acre). Table 26 shows the breakdown of projected maintenance costs for the Urban Lake at Meadowvale West. The costs include allowances for algae control, sediment removal, and aeration. Per unit area, the costs were less than the corresponding unit area cost for maintenance of parkland [24]. It was concluded that man-made lakes were comparatively inexpensive to maintain and may be favoured to complement parkland based on considerations of aesthetics, amenity, and lower cost.

5.3.4 Applications of storage

This section describes storage facilities in Canada and the United States. The material includes examples of facilities for both storm water and combined sewer overflow, and for in-line and off-line storage.

Meadowvale West Lake, Mississauga [24,31]. Construction began in 1976 on an earth dam and silting basin facility to control storm water runoff.

TABLE 23. CAPITAL COST OF STORAGE FACILITIES [29]

Facilities	Capacity 1000 m^3	(mil gal)	Capital Cost $\$/m^3$	(\$/gal)	Operation and Maintenance Cost \$/year	
Storage Reservoirs						
Hillside park	43.1	(11.1)	2.64	(0.01)	---	Earthen Basin
Heritage Park	138.0	(36.4)	2.64	(0.01)	---	Earthen Basin
Oak Lawn	29.5	(7.8)	5.28	(0.02)	---	Earthen Basin
Middle Fork North Branch	740.0	(195.5)	5.28	(0.02)	---	Earthen Basin
Wilke-Kirchoff	123.0	(32.6)	7.93	(0.03)	---	Earthen Basin
Melvina Dutch	204.0	(53.8)	7.93	(0.03)	---	Earthen Basin
Oak Hill Park	95.0	(25.1)	5.28	(0.02)	---	Earthen Basin
Dolphine Park	204.0	(53.8)	2.64	(0.01)	---	Earthen Basin
Average	197.0	(52.1)	5.28	(0.02)		
Storage Tanks						
Cottage Farm, Boston	4.9	(1.3)	1376.50	(5.21)	2 700	Covered
Spring Creek, New York	37.8	(10.0)	615.60	(2.33)		Concrete Tanks
Chippewa Falls, Wisconsin	10.6	(2.8)	76.60	(0.29)	71 000	Covered Concrete Tanks
Humboldt Avenue, Milwaukee	15.1	(4.0)	145.30	(0.55)	55 000	Asphalt Paved Basin
Seattle, Washington	121.0	(32.0)	66.05	(0.25)		Covered Concrete Tanks
Whittier Narrow, Columbus	15.1	(4.0)	450.00	(1.70)		In-Line
Average	34.1	(9.0)	454.43	(1.72)		Open Concrete Tanks
Equalization Basins for Dry Weather Sewage Treatment Plants	3.8	(1.0)	58.12	(0.22)		Earthen Basin
	11.4	(3.0)	26.42	(0.10)		Earthen Basin
	37.8	(10.0)	15.85	(0.06)		Earthen Basin
	3.8	(1.0)	103.04	(0.39)		Concrete Basin
	11.4	(3.0)	73.98	(0.28)		Concrete Basin
	37.8	(10.0)	66.05	(0.25)		Concrete Basin

TABLE 24. CAPITAL COST AS A FUNCTION OF STORAGE VOLUME [29]

Type	Equation	Unit Cost @S = 37 850 m^3 (10 mil gal) $\$/m^3$	(\$/gal)
Earthen	$C = 0.025\ S^{0.73}$	\$ 3.44	(\$0.013)
Concrete w/o Cover	$C = 0.350\ S^{3.58}$	\$35.14	(\$0.133)
Concrete w Cover	$C = 0.400\ S^{0.79}$	\$66.05	(\$0.250)

C = capital cost
S = storage volume

TABLE 25. CAPITAL AND OPERATING COSTS - CANADIAN FACILITIES

Location	$1000\ m^3$	mil Igal	Construction Cost $\$/m^3$	Construction Cost \$/gal	Annual Operation and Maintenance Costs \$/hectare	Annual Operation and Maintenance Costs \$/acre	
Storage Reservoirs							
Bridlewood Manor (1976) (Nepean Township) [69]	4.9	1.3	15.85	0.06	NA	NA	Earthen Basin
Kennedy-Burnett (1976) [70]							
Reservoir	12.1	3.2	31.70	0.12	NA	NA	Earthen Basin
Transport Channel	3.4	0.9	13.20	0.05	NA	NA	
Winnipeg (1976) [32,33]	NA	NA	5.28	0.02	371*	150*	Earthen Basin
Meadowvale (1976) [24,31]	126.0	33.3	15.85	0.06	1976	800	Earthen Basin
Upper Canada Mall (1976) [36]	6.1	1.6	23.78	0.09	NA	NA	Earthen Basin
Storage Tanks							
Halifax (1965) [71]	3.4	0.9	121.53	0.46	NA	NA	Concrete Covered
Dartmouth (1971) [71]	0.76	0.2	330.25	1.25	NA	NA	
Welland (1971) [37]	45.46	10.0	13.21	0.05	NA	NA	Earthen Basin (partially concrete-lined)

*Not including periodic removal of silt, see text.

TABLE 26. MEADOWVALE WEST LAKE ROUTINE MAINTENANCE REQUIREMENTS AND COSTS

Periodic Visits by a Supervisor		
i) 4 visits/year x 2 hrs/visit x \$10/hr	\$ 80	
ii) mileage, 4 @ 32 km x 12.5¢/km	16	
Maintenance - Inlet Structure		
Routine - Surface Skimming and Scum Pit Cleaning 24 Visits x 4 hrs/visit x \$30/hr	2 880	
Maintenance - Shoreline		
i) 2 men x 6 days x 8 hrs/day x \$6/hr	576	
ii) mileage, 6 days x 32 km x 18.5¢/km	36	
iii) materials (stone) allow	300	
Chemical Treatment (for algae control)		
i) labour, 1 treatment/year 1 day x 8 hrs x 2 men x \$6/hr	96	
ii) Chemicals 1 treatment	1 500	
iii) Allowance for boat	125	
Silting Pond - Silt Removal, allow (Based on the accumulation of 2.5 cm/year and removal once every year; approximately 95 m^3 @ \$16.00/m^3)	1 500	
Aeration Equipment Maintenance and Operation, allow	50C	
Make-Up Water 76 m^3 x \$0.17/m^3 x 50 days	650	
Contingencies	1 000	
TOTAL	9 134	Use 9200
Cost/hectare =	\$1 980	

The surface area of the lake at normal water level is 5 ha (12 acres), approximately 5% of the total drainage area of 107 ha (265 acres). The catchment plan and physical data are presented in Figure 42 and Table 27. The bank slope at the shore is 4:1 and the depth at normal water level is approximately 2 m (6 ft) with an additional 1 m (3 ft) available for live storage to maximum water level, which will be reached during the regional storm. Hence, this permanent lake provides 150 000 m^3 (5.3×10^6 ft^3) of permanent storage and 28 000 m^3 (1.0×10^6 ft^3) of active storage, which is approximately equivalent to a depth of 2.54 cm (1 inch) over the drainage area. A drop-inlet spillway controls outflow, with a design flow based on the regional storm.

The primary purpose of the lake is its scenic and recreational potential, i.e., for non-body contact activities such as boating, fishing, etc. The detention facilities have the potential to reduce peak flows to preurbanized levels for storms with return periods of less than one year. As designed and constructed, the lake does not effectively control low flows. Two computer models, SWMM and STORM, were used to simulate urban runoff phenomena for pre- and post-development conditions. Improvement of runoff quality was predicted in comparison with the uncontrolled condition, using a special subroutine of the STORM model to simulate the performance of the settling basin-lake system.

<u>Storm Water Retention Basins, Winnipeg [32,33]</u>. Storm water retention facilities in Winnipeg were designed to provide in-line storage for control of peak flow rates, and for public or private recreational use [32]. Both 'dry' (between storms) and 'permanent' (always partially filled) storage facilities have been built, but it has been recommended that all future facilities be permanent. Ponds were excavated on land bought from developers.

Existing facilities include:

Large Dry-Bottom Basin - This 10 m deep, 12 ha facility (30 ft, 29 acres) is designed to service 525 ha (1300 acres) of residential land through pumped discharge. The basin has 235 000 m^3 (8.3×10^6 ft^3ft) of storage and consumes only 2% of the service area due to its depth. This facility is fenced, and is used solely for storm water retention.

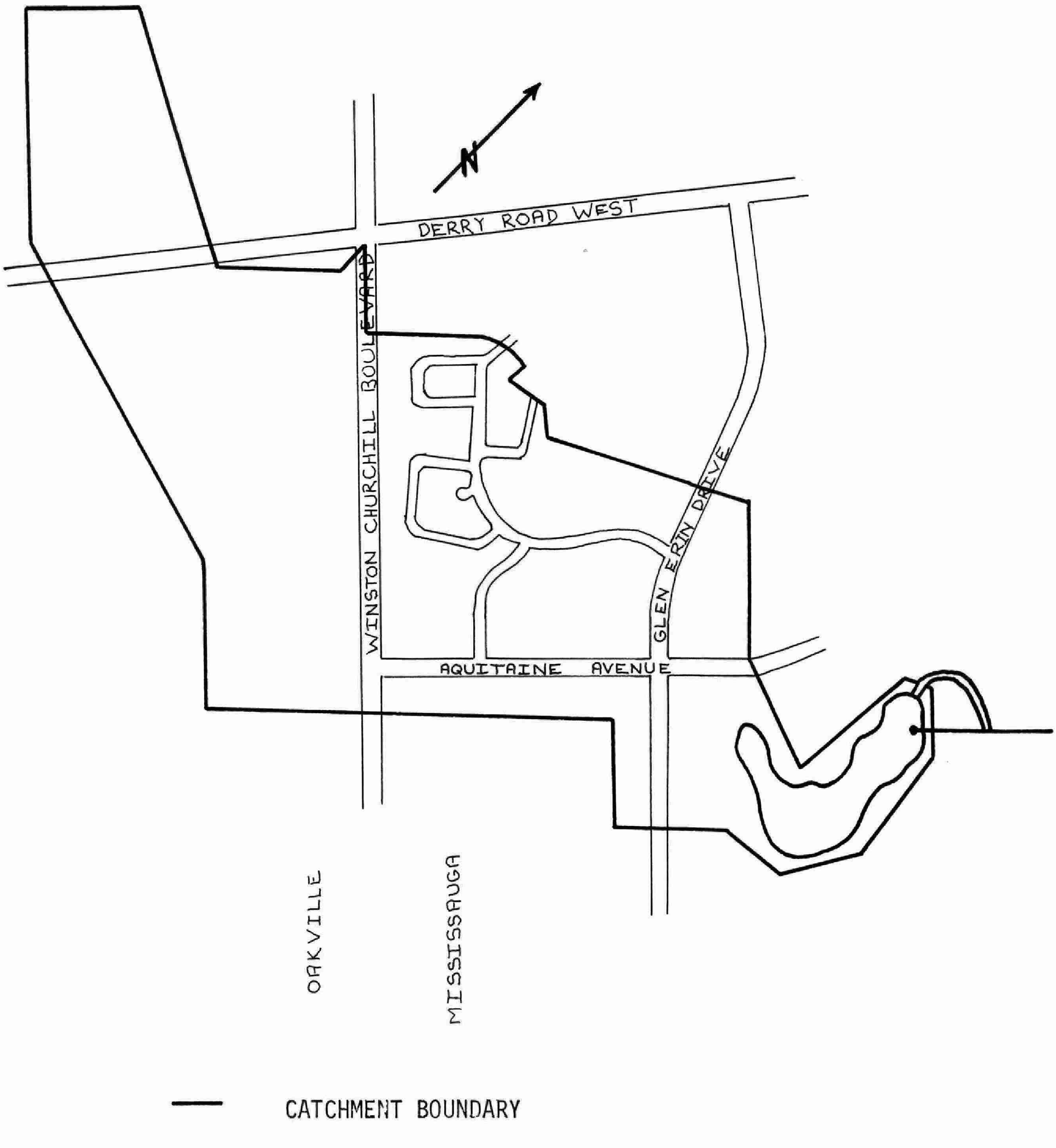


FIGURE 42. MEADOWVALE WEST LAKE AND CATCHMENT AREA [31]

TABLE 27. MEADOWVALE WEST LAKE - CATCHMENT DATA FOR SIMULATION MODELLING [31]

1. Undeveloped Conditions

Non-Urban Watershed Data

Land Area	= 241 acres (97 hectares)
Runoff Coefficient	= 0.25
Depression Storage	= 0.1 in (2.5 mm)

Urban Watershed Data

Land Area	= 1 acre (0.4 hectares)
Land Use	= Single-family residential
Runoff Coefficient	
(pervious)	= 0.15 (75% of area)
(impervious)	= 0.9
Depression Storage	= 0.05 in (1.3 mm)
Gutter Length	= 2 ft/acre (1.5 m/ha)

2. Developed Conditions

Urban Watershed Data

Land Area	= 244 acres (98 hectares)
Runoff Coefficient	
(pervious area)	= 0.15
(impervious area)	= 0.9
Depression Storage	= 0.05 in (1.3 mm)

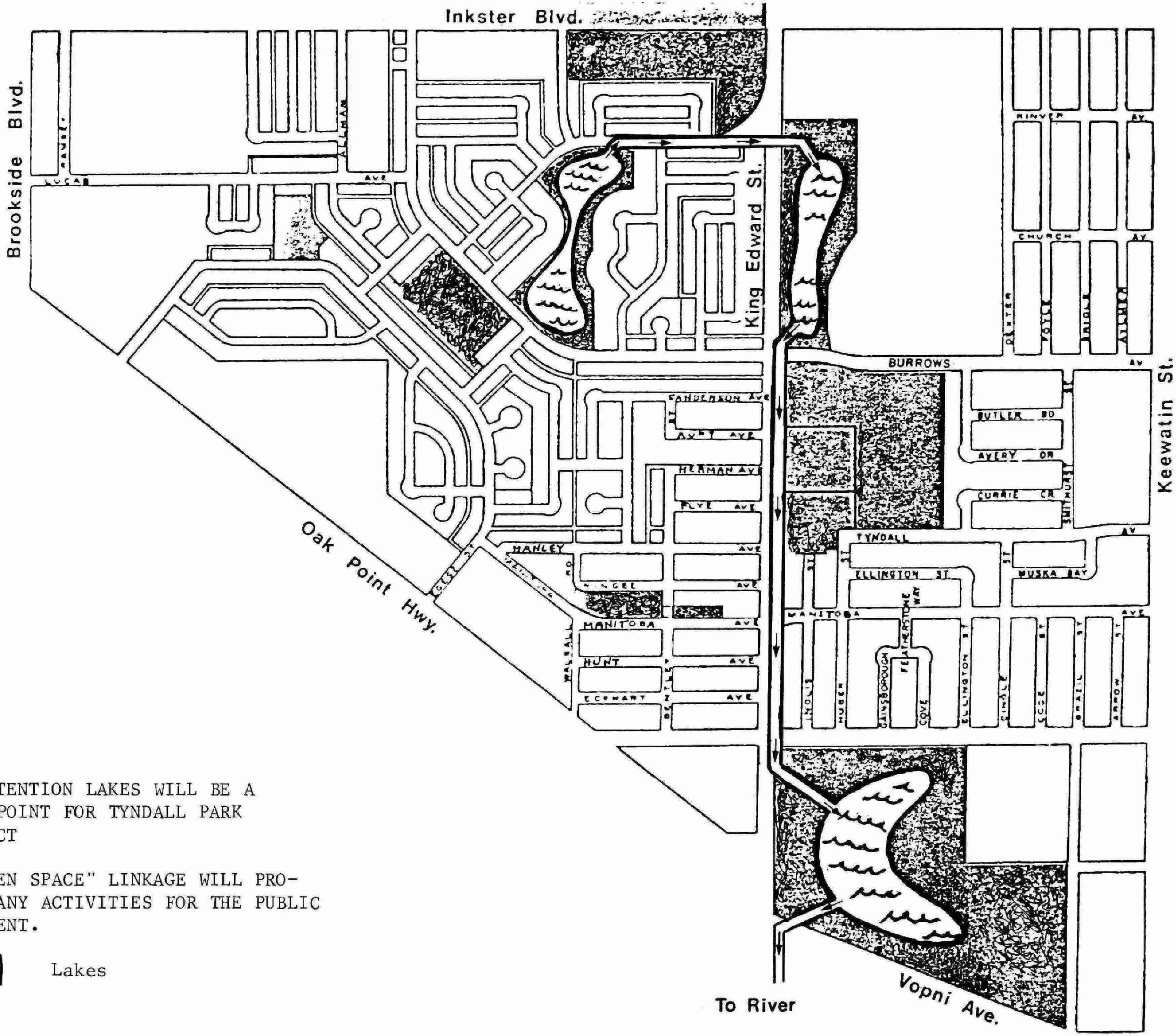
Land Use	% of Land Area	Percent Impervious	Gutter Length m/ha	Gutter Length ft/ac
Single	65.2	30.0	226.0	300.0
Multiple	22.9	65.0	226.0	300.0
Commercial	11.9	85.0	301.2	400.0

Small Dry-Bottom Basins - In 1969, three small off-line detention pits were constructed to permit development of an otherwise undrainable land holding in the St. James area. The lakes operate to attenuate peaks whenever the existing system becomes surcharged. They have not been fronted by private property and vandalism has been a problem. Installation of a tennis court in one of the dry basins has been proposed. In general, these dry bottom basins are considered too small for viable multi-purpose use.

Permanent Lakes - There are presently six permanent retention facility systems in varying stages of development; three of these systems are functional. The Southdale system services a 229 ha (565 acre) subdivision with seven lakes, ranging in area from 1.2-4 ha (3-10 acres). The total lake area is 13 ha (32.3 acres). The Waverley Heights system is a three-lake chain, with surface areas ranging from 1.6 to 2.8 ha (4-7 acres). The Baldry Creek System is a widened out section of an existing rural waterway, consisting of lakes of 1.0 and 2.2 ha (2.5 and 5.5 acres) permanent water surface area, servicing 100 ha (247 acres) of land.

The City of Winnipeg recommends that the slope of embankments be 7:1 within the operating water level range of the ponds, which is generally in the order of 1.2 m (4 ft) plus 0.6 m (2 ft) free-board. The slopes are normally sodded, and recommended slope stabilization includes a layer of crushed stone on top of sand. The city also prefers that an impoundment should not be smaller than 2 ha (5 acres) in size and should occupy a minimum of three percent of the total drainage area. All the lake systems are integrated within residential subdivisions with varying proportions of public shoreline [32]. Figure 43 shows the arrangement of a system of lakes in the Tyndall Park district.

A year-long study of the Southdale and Baldry systems was undertaken in 1975 to assess treatment efficiency, recreational potential, and the distribution and quality of sediments [33]. The true annual treatment efficiency provided by the retention facilities was not determined due to insufficient data collection. Data were scant because of a severe reduction in rainfall occurred over the study period. Based on data for influent and effluent concentrations, it was concluded that the retention basins were providing some level of treatment. The annual pollutant reductions in the Southdale system were estimated as 33% of BOD_5, 79% of SS, and 27% of total P.



THE RETENTION LAKES WILL BE A FOCAL POINT FOR TYNDALL PARK DISTRICT

A "GREEN SPACE" LINKAGE WILL PROVIDE MANY ACTIVITIES FOR THE PUBLIC ENJOYMENT.

Lakes

FIGURE 43. ARRANGEMENT OF LAKES - TYNDALL PARK DISTRICT, WINNIPEG [32]

The bacteriological water quality was designated suitable for primary contact recreation in the Southdale system. In the Baldry Creek system, where sanitary cross-connections were believed to exist, the bacteriological water quality was suitable for primary body contact 24 hours after a major rainfall event. Designation for primary contact was based on the assumption that the biological quality would not differ significantly under disturbed water conditions. The quality of water in both the lakes was suitable for secondary recreational uses at all times.

However, the biological water quality was found to be unacceptable for primary contact recreation in both Southdale and Baldry. Extensive weed and algae control would be necessary to meet acceptable standards.

The average rate of sedimentation was 1.3 cm (0.5 in) per year and dredging would therefore be necessary after about 25-50 years. Dredging would also be necessary to remove sediment buildup in the vicinity of storm sewer inlets to the lakes. The sediment had an average lead content of 80 mg/kg, the chief source of which was identified as automobile exhaust. High chloride concentrations were also present due to heavy use of salts for road deicing. The levels of lead and chlorides were not deemed to be harmful.

This storage alternative has cost advantages of about one-sixth over the cost of conventional piping systems, with the added benefits of recreational potential and some treatment of runoff [35].

Undergound Detention Tanks, Scarborough, Ontario. The Borough of Scarborough in Metropolitan Toronto was faced with a basement storm water flooding problem in a separated sewerage area in which roof leaders were connected to the storm sewers.

Centralized downstream detention was investigated but would not eliminate the surcharging due to inadequate hydraulic capacity in upstream existing sewers. The cost for this approach was estimated at $400 000 or $530/house.

Decentralized detention was investigated and found likely to eliminate basement flooding, even for major storms, at a cost of about $200/house.

A plan is being implemented whereby road runoff is held temporarily in underground detention tanks [35] placed between the catch

basins and the street storm sewers. The outlet rate from storage is controlled by a proprietary constriction device, the Hydrobrake. a typical system arrangement is shown in Figure 44B. The detention tank holds the road runoff from most storms allowing peak intensities to pass before filling the tank completely. In an extreme or very prolonged event, road ponding may eventually occur.

Figure 44A is a detailed drawing of a typical Hydrobrake. The device is claimed to be virtually clogfree when protected by an inlet grating on the street, since it imparts a strong swirling motion to the storm water. Figure 44C shows the typical head/discharge relationship of a Hydrobrake. The curves indicate that significant flow commences only when a predetermined head has been reached.

Ten small detention basins, each with a Hydrobrake, were proposed. Four of these units were installed in the fall of 1976. The cost of equipment and installation for the four units was approximately $50 000. Six more units were to be installed by 1978. All the hydrobrakes in Scarborough have a discharge opening of 20 cm (8 in) which provides a flow of 56 L/s at a peak head of 200 cm (80 in).

<u>Storm Runoff Detention Ponds, Blu-Aire Estates, Township of Sarnia [36].</u> Blu-Aire Estates (70 ha) is located in the Township of Sarnia (Figure 45) in an area where flat topography, a high water table in sandy soil, and two connected abandoned gravel pits provided interesting opportunities for in-line storm water storage development.

The Township was not willing to spend the large sum of money needed to deepen and widen existing drains to accommodate surface runoff in the conventional manner; and the developers would not have an economically viable project if they had to absorb the total cost of conventional drainage.

A consulting firm was engaged to study alternate means of storm water drainage. They suggested the use of the gravel pits as detention ponds and, after examination of inflow/outflow relationships and volumes of runoff, a hydrograph simulation model was used to design the sewer system and analyse the detention capabilities of the gravel pits.

The following design criteria were used:

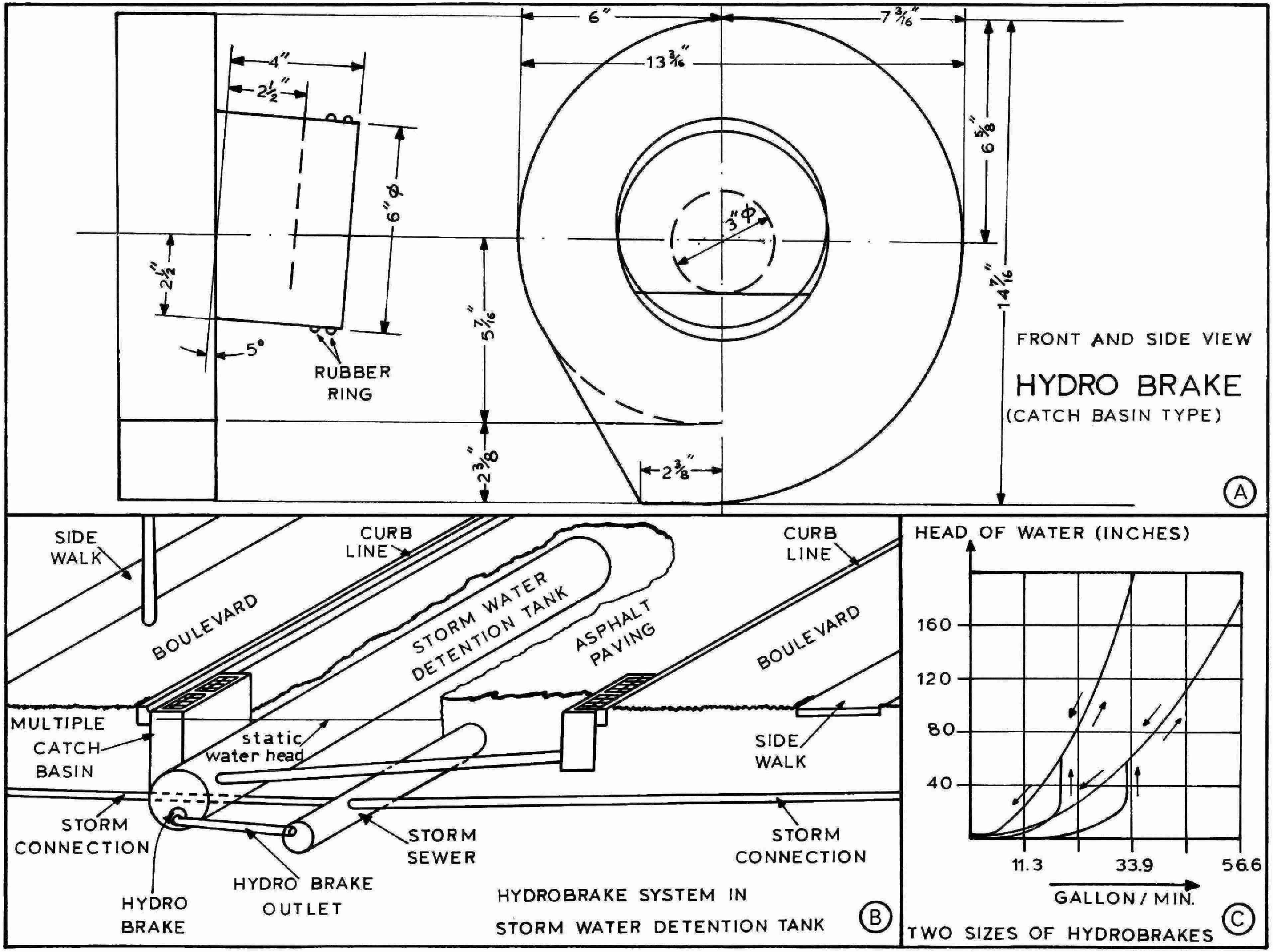


FIGURE 44. STORM WATER DETENTION TANKS AND HYDROBRAKE, SCARBOROUGH, ONTARIO

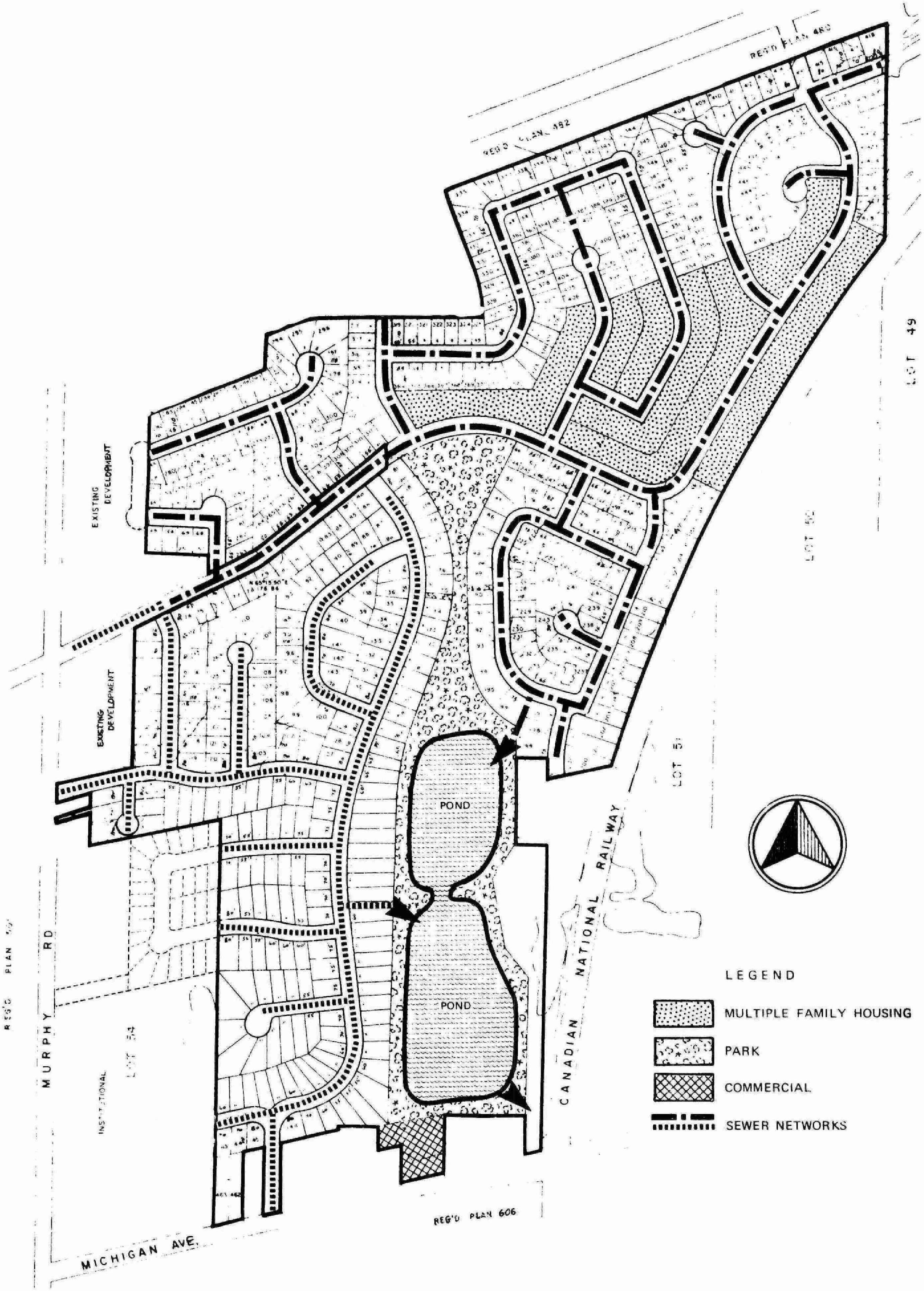


FIGURE 45. STORM DRAINAGE AND LAND USE AT BLU-AIRE ESTATES, TOWNSHIP OF SARNIA [36]

Storm sewers: Rational Method for a two-year rainfall intensity curve and a 20-minute inlet time.

Detention ponds: The Colorado urban hydrograph method was used. Rainfall/runoff relationships were developed for two-year and 100-year frequency storms.

A wide range of values for sewer grades, storm duration, and precipitation/runoff was examined in detailed calculations. In the selected design, the north trunk storm sewer discharges storm water into the north pond and the south trunk into the south pond. Peak discharge rate to the municipal sewer was calculated to be 0.05 m^3/s (2 cfs). A discharge of 2.3 m^3/s (80 cfs), would have been expected without using the gravel pits for storage.

As a result of the controlled storm water discharge, minimal alteration was required to the existing municipal storm drain to accommodate the extra flow from the sewer after development.

Maximum storage requirements for a two-year storm and 100-year storm were found to be 9200 m^3 and 16 550 m^3 respectively (325 000 ft^3 and (585 000 ft^3) with depths of 7.5 m (25 ft) and more in some areas, the capacity of the ponds was found to be sufficient for a high-intensity storm. The proposed facility would cost $190 000 with the cost of alternative methods of routing storm water to Lake Huron being in the order of $585 000. The water quality is expected to be good enough to support a population of fish and plans call for recreational development around the ponds.

<u>Retention Basin for Combined Sewer Overflow, Welland, Ontario [37]</u>. In 1972, the City of Welland completed construction of a 2-ha (5-acre) off-line holding basin for combined sewage with a capacity of 45 460 m^3 (10 million gallons) at a cost of $400 000. Previously, combined sewer overflows had been discharged to Lyons Creek, but construction of the new Welland Canal cut off this outlet. The retention basin was the lowest cost alternative available.

The basin provides retention for wet weather flows in excess of intercepter capacity and the combined sewage is returned to the WPCP at periods of low flow. The limiting factor on drainage rate is the WPCP capacity - returned flow receives full secondary treatment.

Tank discharge is by gravity flow through a 90 cm (36 in) diameter sewer to a pumping station, and outflow rate is controlled by adjusting a motorized gate remotely from the WPCP or manually at the basin itself. Discharge rate from the tank is measured through a Parshall flume equipped with a daily totalizer. Any overflow from the basin discharges via an open ditch to the Welland Canal. Overflow volume is not measured. The relative locations of retention basin, pumping station, Welland Canal and interceptor sewer are shown in Figure 46A.

The basin is of irregular shape with overall bottom dimensions of 23 m x 76 m x 122 m x 152 m (75' x 200' x 400' x 500') (Figure 46B) with a maximum working level of 4.3 m (14 ft) and side walls with a slope of about 1:5 (Figure 46C).

The basin has two cells in series. Cell #1 is concrete lined; cell #2 is sodded. There is a baffle wall between cells #1 and #2 designed to retain floatables and to contain small overflow in cell #1. The bulk of the solids entering the tank settle in cell #1. This cell is cleaned after draining by flushing settled solids to a drainage trough with the aid of a truck-mounted water cannon. Although the basin is located close to housing, no odour complaints have been received to date.

The basin originally served a 182-ha (450-acre) area, which has now been reduced to about 135 ha (338 acres) by ongoing sewer separation. The population, with 95% residential and 5% industrial land use, is about 4000. Volume captured during 1976 was in the order of 159 000 m^3 (35 x 10^6 gallons) corresponding to 11.4 cm (4.5 inches) precipitation or about 12% of the estimated dry weather flow from this area. Tank overflow used to occur two or three times per year, but with a reduced service area, the frequency of overflow is also reduced.

<u>Storm Drainage Retention/Recharge at Northville Heights, Town of Paris [38]</u>. The 24-ha (60-acre) Northville Heights subdivision in the Town of Paris has been designed with a storm water retention/recharge pond as an integral part of the storm water drainage system. There was insufficient capacity in the existing local storm sewer system to deal with more than about 2% of the runoff from Northville Heights. This, coupled with the granular subsoil and almost totally residential nature of the development suggested a recharge pond as a possible means of storm water disposal.

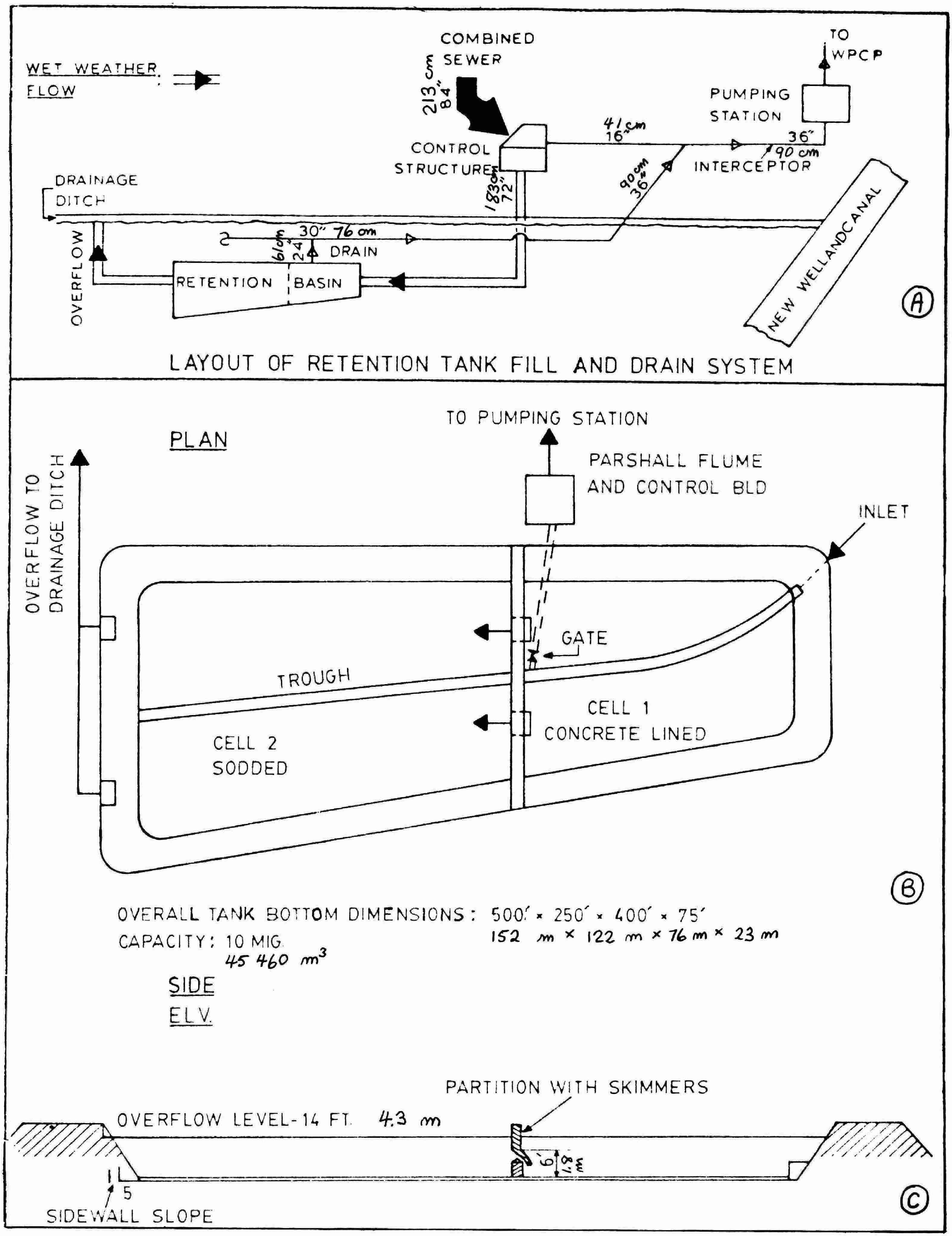


FIGURE 46. WELLAND CSO RETENTION BASIN [37]

Groundwater pollution from the recharge pond will be minimized by diversion of the initial storm flow to the local storm sewer system of an adjacent area. This will be achieved by means of a control structure in a manhole and an associated control section of storm sewer constructed in such a manner that storm water will only begin to flow to the recharge pond when the available hydraulic capacity in the local storm sewer has been utilized. The first flush of polluted water will therefore be directed away from the pond.

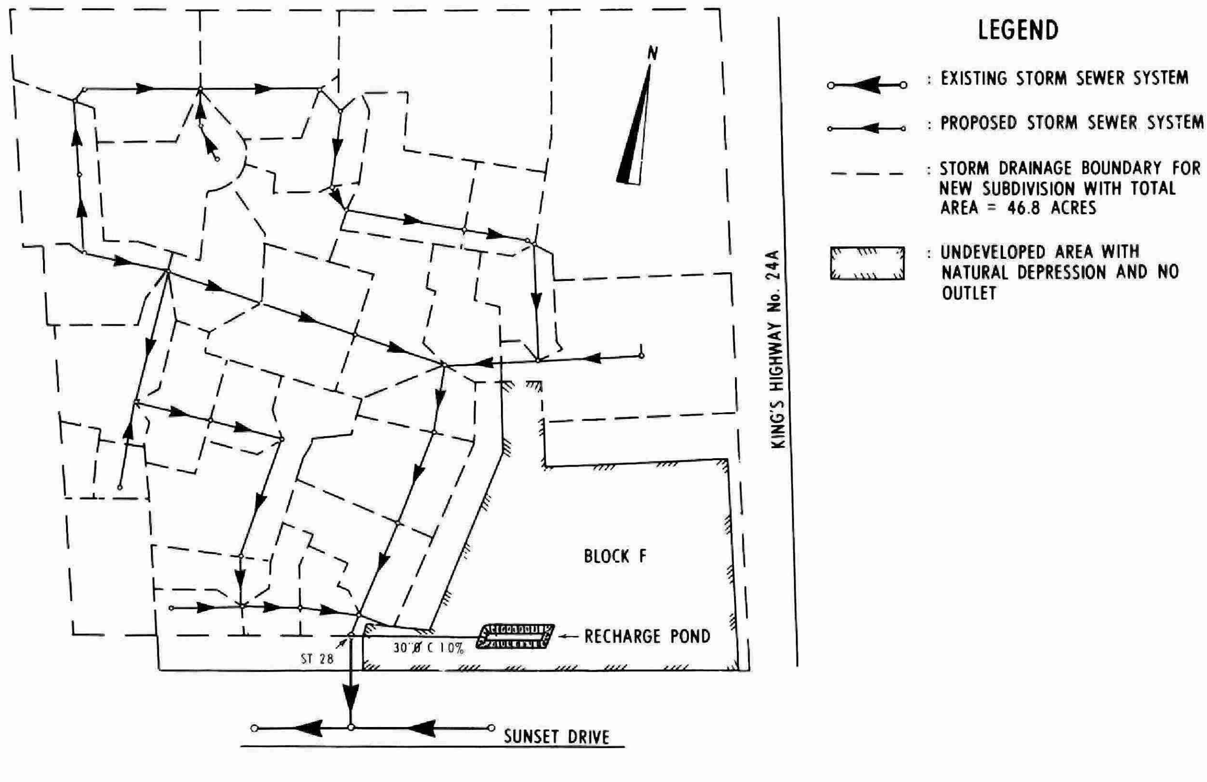
At the end of major storms, a portion of water remaining in the pond will backflow into the adjacent storm sewers until the falling water level in the pond equals the overflow weir elevation in the control structure. Infiltration from the pond is expected to dispose of the stored volume. Within the pond itself silty overburden has been stripped to expose the coarse granular subsoil, which has a permeability of 13 mm/h (0.52 in/hr).

Storm water flow to the sewers within the subdivision will also be reduced because of the porous nature of the soil, the relatively low percentage of impervious area, and because the roof leaders discharge onto the ground. Wherever possible, lot grading has also been designed to delay the contribution of runoff to the sewer system.

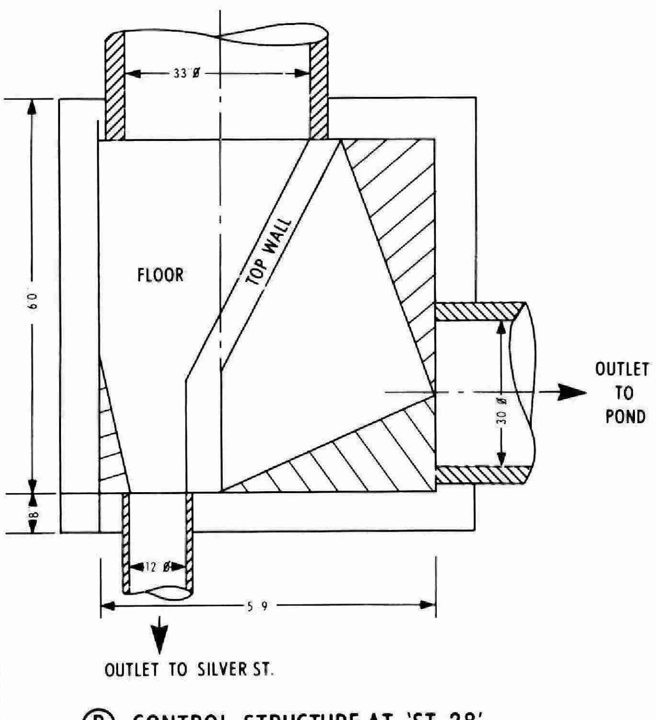
The design of sewers was based on the Rational Method using a five-year storm. Analysis of off-line storage and pond dimensions was carried out using a model developed by the consulting engineer and patterned after the storm water management model (SWMM). Three storms were investigated: a five-year storm, a three-times-per-year storm, and the Ont-8-69 storm (25.4 mm per hour for approximately six hours).

A pond capacity of 990 m^3 (35 000 ft^3) was sufficient to accommodate the three-times-per-year and the five-year storm. The Ont-8-69 storm resulted in a runoff to the pond of 3.3 times the value for the five-year storm and indicated that approximately 1980 m^3 (70 000 ft^3) of additional storage would be required. A low-lying area to the north of the potential pond site was found to adequately and safely provide this extra storage volume.

Overall site layout, including storm drainage, is shown in Figure 47A. Figure 47B shows details of the control structure.



Ⓐ SITE LAYOUT AND STORM DRAINAGE



Ⓑ CONTROL STRUCTURE AT 'ST 28'

FIGURE 47. STORM DRAINAGE AND RETENTION-RECHARGE POND AT NORTHVILLE HEIGHTS, TOWN OF PARIS [38]

The pond bottom dimensions are 5.5 m x 48 m (18' x 158') with side slopes of 3:1. The pond has a bottom elevation of 252 m (827 ft), an overflow berm on the north side with top elevation of 254 m (833 ft), and a retaining berm on the south side with a top elevation of 255 m (836 ft). Thus, in heavy storms, water would overflow into Block F from the north side. Under the subdivision agreement, Block F was designated parkland.

The responsibility for operating and maintaining the recharge system and the storm water retention facility has been accepted by the Town of Paris.

The pond should be aesthetically unobtrusive, especially after development is complete and will be fenced since recreational potential is not a design consideration.

5.4 Treatment

5.4.1 Characterization for Treatment

It is useful to compare the characteristics of storm water and combined sewer overflows (CSO) to those of sanitary sewage to indicate differences and similarities which might affect treatment.

Table 28 shows the concentrations of major constituents in medium-strength sanitary sewage. The dissolved and suspended matter in sewage is both organic and inorganic. The organic content is derived mostly from human and household wastes, including paper, grease, vegetables and synthetic materials, and the resultant BOD is generally between 100 and 300 mg/L. Most of the suspended matter is organic and flocculent in nature. Inorganic material in sewage can include clay, sand, silt, and dissolved salts such as the sulphates, carbonates, bicarbonates, and chlorides of calcium, magnesium, sodium, potassium, and iron. These are normally not present in amounts troublesome to sewage treatment processes [39]. The two most common nutrients are nitrogen and phosphorus. Domestic sewage normally contains concentrations of both which are greater than required for efficient biological treatment.

Waste strength and solids concentrations vary from hour to hour and between different sewerage systems. The quantity of sewage usually varies throughout the day with peak flows around meal times and low flows

TABLE 28. QUALITY OF TYPICAL MEDIUM-STRENGTH SANITARY SEWAGE

Parameter	Concentration mg/L
BOD	100 - 300
Total Solids	500
Total P	6 - 10
Total Volatile Solids	350
Total Fixed Solids	150
Total Suspended Solids	300
Total Dissolved Solids	20
NH_4-N	15 - 50
Organic N	25 - 85
NO_2-N	<0.1
NO_3-N	<0.5
Total Coliforms	10^7 - 10^8 organisms/100 mL

at night. Typically, WPCP's have a hydraulic capacity of 2.5 to 4 times the average dry weather flow (DWF) and provide full biological treatment to about twice the average DWF.

Both the quantity and quality of storm flows are much more variable than those of sanitary sewage. Within a single catchment, storm flows vary widely during a single storm event and between storm events. Differences in climate, soil characteristics, land use, topography, sewerage, and drainage practices all contribute to this variability.

Flow-weighted annual average concentrations of raw sewage, sewage treatment plant effluent, combined sewer overflow, and storm water as derived in a recent Canada-Ontario study [40], are presented in Table 29 for four key pollutant parameters. The concentrations shown are considered representative of Ontario communities discharging to the Great Lakes.

Storm water and CSO both have suspended solids concentrations comparable to raw sewage. BOD and total phosphorus concentrations in CSO are above those of WPCP final effluents. Although not noted in the table, average concentrations of total coliforms in both types of storm

TABLE 29. AVERAGE ANNUAL CHARACTERISTICS OF RAW AND TREATED SEWAGE AND STORM FLOWS FROM ONTARIO COMMUNITIES DRAINING TO THE GREAT LAKES [40]

	Raw Sewage	WPCP Final Effluents*	Combined Sewer Overflows**	Storm Water**
BOD_5	165	20	41	14
SS	225	26	190	170
Total N	30	18	8.3	3.5
Total P	6.5	1.0	1.4	0.35

All concentrations in mg/L

* Weighted to reflect discharges from both primary and secondary treatment plants.
**Weighted to reflect overall land use.

flow are typically at least an order of magnitude higher than disinfected sewage effluent concentrations.

In Ontario, primary treatment of sewage is mandatory. Higher levels of treatment, such as secondary treatment, advanced biological treatment, etc., are imposed as required to maintain receiving water quality for intended uses.

Treatment of storm water or CSO will become cost-effective when a high level of sewage treatment is achieved, or if "wet-weather" violations of water quality criteria are more frequent or more significant than "dry-weather" violations. The characteristics of each type of storm flow are discussed further in the following sections.

5.4.1.1 Characteristics of storm water. Table 30 presents the range of concentrations of various parameters in storm water. The sources of storm water pollutants are diverse and include dustfall, soil erosion, litter, and chemicals applied to land (Table 31).

The suspended solids in storm water are significantly different from those in sanitary sewage. Eroded soil material can range from 0.02

TABLE 30. VARIATIONS IN QUALITY OF STORM WATER AND COMBINED SEWER OVERFLOWS*

Parameter	Concentration** Storm Water	Combined Sewer Overflows
Total Solids	199 - 14 600	20 - 8 760
Suspended Solids	2 - 36 250	20 - 2 000
Settleable Solids	0 - 7 640	0 - 1 550
Total Volatile Solids	0 - 1 110	10 - 1 280
Total Coliforms (counts/100 mL)	$2 \times 10^2 - 1.5 \times 10^7$	$2 \times 10^3 - 9 \times 10^6$
Fecal Coliforms (counts/100 mL)	$10 - 1.12 \times 10^7$	$2 \times 10^3 - 1.7 \times 10^7$
Fecal Streptococci (counts/100 mL)	$3 \times 10^3 - 6 \times 10^4$	$2 \times 10^3 - 2 \times 10^6$
BOD_5	0.5 - 700	11 - 685
COD	5 - 3 100	13 - 1 760
Total Phosphorus as P	0 - 9.4	0.8 - 9.4
Soluble Phosphorus as P	0.002 - 0.16	0.1 - 6.2
Total Nitrogen	0.5 - 11.8	1.0 - 16.5
Nitrate Nitrogen as N	0.2 - 2.9	-
Ammonia Nitrogen as N	0.2 - 3.0	0.1 - 12.5
Kjeldahl Nitrogen as N	0.5 - 7.5	-
pH (units)	5.3 - 8.7	4.9 - 8.7
Grease and Oil	-	0 - 400

* Range of mean values given in various studies.
**Concentrations in mg/L except as noted.

TABLE 31. POLLUTANTS IN URBAN RUNOFF

- Colour causing materials
- Turbidity
- Foam causing materials
- Floating material
- Street litter debris
- Material from street or pavement surface
- Debris from vacant lands
- Ice control chemicals
- Pest control chemicals
- Fertilizers
- Droppings from animal or bird sources
- Lawn or garden litter
- Household or commercial refuse
- Air deposited materials from precipitation
- Twigs and leaves
- Paper
- Plastic materials
- Tire and vehicular exhaust residue
- Heavy metals
- Hazardous material spills
- Other large, heavy items

mm to 2 mm in size [41] and from clay to small stones. Street surface contaminants are largely sand [42] in the size range of 0.04-4.8 mm (0.0016-0.2 in), with small proportions of silt and clay. Dustfall generally ranges downward in size from 0.04 mm [43]. Combinations of these materials result in a suspended solids fractions high in non-volatile settleable materials which are discrete and non-flocculent. Clays may require chemical coagulation for effective removal. Larger suspended materials such as leaves and paper are not adequately or effectively sampled in most monitoring programs; hence their presence is not accounted for in most storm water characterizations. These materials are always present to some extent in storm water and in some cases or some seasons may represent a significant fraction of the total suspended matter. Provision for handling these materials should be made in design of treatment processes.

Concentrations of nitrogen and phosphorus are generally lower in storm water than in sewage but still may be significant when compared to WPCP effluents which have been treated to remove phosphorus or to nitrify or remove ammonia. The settleable fraction of the phosphorus in storm water is variable, and chemical coagulation may be needed to achieve high percentage removals. Storm water generally has a low alkalinity and the effects of coagulants on pH should be evaluated during treatability studies.

Storm water discharges contain levels of indicator bacteria well above those considered acceptable for total body contact recreation. Recent studies of storm waters in Ontario have concluded that the runoff resembles dilute raw sewage in microbiological characteristics, although the sources of bacteria in storm water appear to be predominantly animal rather than human. The studies concluded that storm water represents a presumptive public health risk [43].

While human activities such as cooling water discharges contribute to dry weather flows in storm sewers, the 'forcing function' which normally generates high flow rates is precipitation or snowmelt. Consequently, storm water flow rates have greater variability over shorter time periods than sanitary sewage flow rates. Fluctuations in runoff quality are at least partly dependent on the intensity of

precipitation; peak concentrations of pollutants are not necessarily encountered at the start of a storm and marked first flush effects in pollutant concentrations are not always observed.

5.4.1.2 Characterization of combined sewer overflow. Combined sewer overflow (CSO) consists of varying mixtures of sanitary sewage and storm water, plus the products of sewer scour. Table 30 illustrates the variability of CSO quality. The data in the table do not reflect the presence of the gross solids which often appear in combined sewer overflows.

Overflows can be expected to contain a large fraction of readily settleable material which may be septic and odiferous. Table 32 illustrates that, while the first flush of CSO may contain high levels of particulates and other pollutants, extended overflows resemble dilute sanitary sewage.

The characterization, handling, and disposal of residual sludges from the treatment of CSO has received comparatively little attention to date, but the need will become more pressing as greater volumes of overflow are treated and sludge volumes increase. A recent study [46] found that the volume of sludge generated by treatment varied from 1-6% of the processed volume, and that the volatile solids content, calorific value, and heavy metals concentrations all varied widely.

5.4.2 Selecting and sizing wet weather treatment facilities

Because of the quantity and quality of CSO and storm water are highly variable and the 'forcing function' for generating flow is precipitation or snowmelt, the selection and sizing of wet weather treatment facilities necessarily proceeds differently than sizing WPCP's for sanitary wastes.

The generalized approach to selecting and sizing wet weather treatment processes is summarized in Table 33. Steps 1-4 determine the degree of wet weather control required and have been discussed in detail in Chapter 3. This section is concerned with steps 5 and 6 for both combined and storm sewer systems.

5.4.2.1 Combined sewer overflows. The degree of control required for CSO may be expressed in various ways, such as the number of 'significant' overflow events to be permitted per year or the required percentage

TABLE 32. COMBINED SEWER OVERFLOWS: QUALITY OF FIRST FLUSH AND EXTENDED OVERFLOW [45]

Parameter	Concentration (mg/L)	
	First Flush	Extended Overflows
COD	500 - 765	113 - 166
BOD	170 - 182	26 - 53
SS	330 - 848	113 - 174
VSS*	221 - 495	58 - 87
Total N	17 - 24	3 - 6

*Volatile suspended solids.

TABLE 33. SEQUENCE OF STEPS FOR SELECTING AND SIZING TREATMENT FACILITIES

1) Determine the existence of a receiving water problem.
2) Carry out comprehensive analysis of all sources contributing to water quality problems.
3) Place urban runoff in context.
4) Identify the parameters in urban runoff which need to be controlled and required degree of control for each.
5) Select the appropriate type of treatment.
6) Size the treatment facility using an optimization process which also includes the cost of storage.

removal of pollutants for a given volume fraction of annual overflow. For treatment facility design, these requirements usually must be restated as a target or a maximum permissible mass emission of pollutants from a particular storm event or events.

Determining the mass emission of pollutants requires knowledge of untreated CSO or storm water characteristics, treatment process performance efficiency, and treatment process "volumetric" efficiency. Performance efficiency is the percentage removal of pollutants in the portion of the flow that is treated. Volumetric efficiency is the percentage of the total flow that is treated.

Predictions of the quantity of untreated wet weather flow can be obtained from simulation modelling with sufficient accuracy for facility design. However, simulation predictions of untreated wet weather quality are not sufficiently accurate for design purposes and a field sampling program is required to establish a statistically sound characterization of raw wet weather flow quality. A recent EPA publication [47] contains extensive methodology relating to sampling programs in combined sewer systems. Standardized parameters suggested for combined sewer sampling programs are given in Table 34.

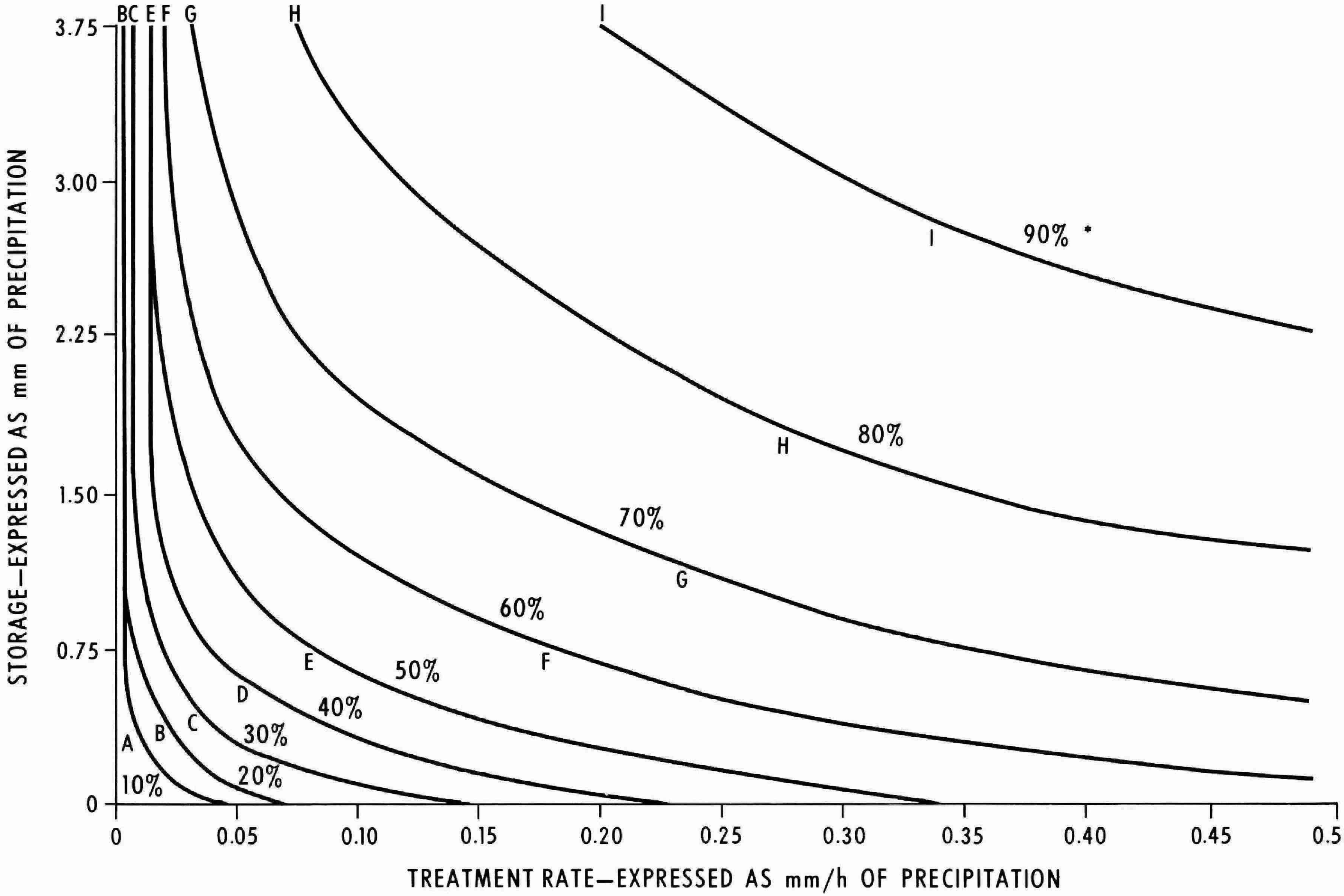
Once design values for quantity and quality of wet weather flow have been derived, the mass of pollutants discharged in untreated flow can be determined for periods or storm events in which overflows occur and control is required. The pollutant mass removal needed in treatment to meet the target values can then be determined.

Different combinations of volumetric efficiency and treatment efficiency may result in the same overall mass removal, i.e., treatment of a high percentage volume of the waste at low efficiency may result in the same pollutant removal as treatment of a lower percentage volume at higher treatment efficiency. The treatment rate or design capacity may be traded off against upstream storage. Heany, Huber, and Nix [48] present a generalized approach to the interrelated costs of storage and treatment. Figure 48 illustrates one method of expressing storage-treatment tradeoffs.

For preliminary design, the treatment efficiency assumed for specialized wet weather treatment processes can be based on the results

TABLE 34. SUGGESTED STANDARDIZED STORM WATER MONITORING PARAMETERS [47]

Regular Parameters	Probable Additional Parameters
Suspended solids	Settleable solids
Total oxygen demand	Volatile suspended solids
BOD_5	Lead
Fecal coliform	Zinc
Nitrite and nitrate	Copper
Kjeldahl	Chromium
Total phosphorus	Mercury
pH	Cadmium
	Arsenic
	Nickel
	Tin
Parameters of Potential Interest	
Asbestos	Odour
Asphalt and road materials	Particle size distribution
Colour	Rubber
Debris and bulk solids	Specific conductance
Density	Sulphates
Specific gravity	Temperature
Solids settling velocity distribution	Trace organics
Oil and grease	



* 90% MASS REMOVAL OF SELECTED POLLUTANT, ASSUMING FIRST FLUSH OCCURS

FIGURE 48. STORAGE-TREATMENT ISOQUANTS FOR PERCENT BOD REMOVAL WITH FIRST FLUSH - TYPICAL PLOT

from the literature. For final design, pilot-plant studies are recommended because of the current lack of Ontario experience with treatment of CSO.

5.4.2.2 Storm water. In newly developing areas, decisions on the need for storm water treatment will generally be made as part of the MOE review process as it applies to new development. Verification of runoff quality from proposed development is obviously not possible. If adjacent catchments exist with characteristics similar to those of the proposed system, runoff or dust and dirt samples can be collected as discussed in Chapter 3 to provide best available default values for calculating loadings. Alternatively, default values obtained in previous studies on similar catchments may be used.

Using flows from simulation modelling, the selection and sizing of treatment facilities to meet target pollutant mass emissions can proceed as discussed previously for combined sewer overflows. For new developments, source controls should be evaluated along with storage/treatment options and the most cost-effective combination of measures should be selected. The release rate of storm water from storage treatment facilities must be compatible with watershed needs. This may place some constraints on treatment process selection.

5.4.2.3 Dual use facilities. Dual use refers to use of the same facilities for the treatment of both wet and dry weather flows from combined sewer systems. One way of accomplishing this is to design new facilities at WPCP's to operate in dual mode, e.g., low-rate trickling filters in dry weather can be run as high-rate trickling filters in wet weather. This type of dual use is only of value when intercepter sewer capacity is adequate to transmit a significant fraction of wet weather flows to the WPCP. Alternatively, wet weather flows intercepted and stored during storm events can be released to the (dry weather) WPCP when it is underloaded, e.g., during weekends or at night. A third possibility is use of the same facility both to store wet weather flows and to equalize dry weather flows. Dry weather flow equalization could increase the efficiency of the WPCP.

The dual use concept may be applicable in some situations and should always be considered. There is a potential for reduced overall

capital investment in pollution control when unit processes for wet weather treatment can be integrated into the dry weather WPCP treatment flowsheet [49].

5.4.3 Wet weather treatment processes

In the past few years, work has been carried out to adapt existing sewage treatment technology to the treatment of wet weather flows. More recently, some newly developed treatment processes devised specifically for the more variable loadings and higher rates characteristic of storm-induced flows have become available.

The following sections briefly describe principles of various processes which have been used successfully at full scale, as either demonstration or permanent facilities. Applications of the processes to CSO and storm water are given in Sections 5.4.5 and 5.4.6, respectively.

Most of the material has been extracted from reports on EPA sponsored projects. Two EPA documents are particularly valuable references on treatment technology [5,49].

5.4.3.1 Physical processes. The following physical treatment processes are applicable to wet weather flows:

- sedimentation,
- screening,
- filtration, and
- regulators/concentrators.

In conventional sewage treatment, sedimentation is achieved in on-line settling tanks, for which theory and practice are well developed. Direct application of sewage treatment practice to treatment of storm flows is not cost-effective because such flows are intermittent and variable in nature. Storage basins, which also provide an opportunity for settling, are a cheaper alternate means of achieving sedimentation. The basins are operated so that they are empty or only partly full at the start of a storm event. When the available storage capacity is filled, flow can still be routed through the storage basin to achieve some settling.

In flow-through designs, hydraulic detention periods as low as 10-20 minutes have been used at peak design flow. Detention time is

correspondingly greater at lower flows. The minimum detention period in many facilities has been based on the contact time required for chlorine disinfection. In such cases, the chlorine addition point is located upstream of the storage tank to maximize the contact time without constructing a separate chlorine contact chamber.

Storage/sedimentation basins are most cost-effective when the entire tank contents, including sludges, can be released to a WPCP between storm events. Sedimentation is one of the most common forms of treatment provided to wet weather flows.

Screening is a process which separates solids from liquids using a woven fabric, screen, or sieve plate. The solids retained on the screen are removed in the form of a sludge, concentrate, or cake. Pollutant reduction depends on the type of screen, the size of the screen openings, and the size distribution of suspended solids in the wastewater.

Screening devices used in wastewater treatment include bar screens, coarse screens, fine screens, and microstrainers. Bar screens and coarse screens have openings in the range of 2.5-7.5 cm (1-3 in) and 0.5-2.5 cm (0.2-1.0 in), respectively. They are used as protective devices to remove gross suspended matter, rather than to produce significant pollutant removals.

Fine-media screens and microstrainers generally have apertures in the range of 75-1600 microns and 23-65 microns, respectively, although there is some overlap in aperture sizes for each type. Both types can produce significant pollutant reductions and both have been evaluated extensively in the treatment of CSO. Table 35 provides a summary description of rotary and static screen types and their applications.

In some instances pollutant removals have been enhanced by addition of coagulants to increase the effective particle size of the suspended matter. Removal of particles which are smaller than the screen opening is obtained by features of screen design which promote mechanical agglomeration or straining.

Generally, suspended solids removal may be increased by reducing the size of the screen aperture and/or the surface loading rate. At higher percentage removals, reduced hydraulic capacity and blinding of screen media may become critical factors in process performance and

TABLE 35. DESCRIPTION OF TYPES OF FINE MESH SCREENING DEVICES USED IN COMBINED SEWER OVERFLOW TREATMENT [49].

Type of Screen	General Description	Application	Comments
Drum Screen	Horizontally-mounted cylinder with screen fabric aperture in the range of 100 to 841 microns. Operates at 2 to 7 rpm.	Pretreatment	Solids are trapped on inside of drum and are backwashed to a collection trough.
Microstrainer	Horizontally-mounted screen cylinder. Screen fabric aperture 20 to 65 microns. Operates at 2 to 7 rpm.	Main Treatment	Solids are trapped on inside of drum and are backwashed to a collection trough.
Rotostrainer	Horizontally-mounted cylinder made of helically wound wedge wire. Aperture in the range of 250 to 2500 microns. Operates at 1 to 10 rpm.	Pretreatment	Solids are retained on drum surface and removed by a scraper blade. Disposable sludge possible.
Disc Strainer	Series of horizontally-mounted woven wire discs mounted on a centre shaft. Screen aperture in the range of 45 to 500 microns. Operates at 5 to 15 rpm.	Pretreatment or Main Treatment Thickening Device	Unit achieves 12 to 15% solids cake.
Rotary Screen	Vertically-mounted drum. Screen fabric aperture in the range of 74 to 167 microns. Operates at 30 to 65 rpm.	Main Treatment	Splits flow into two distinct streams: effluent and concentrate in the proportion of about 70:30.
Static Screen	Stationary inclined screening surface. Wedge wire mounted perpendicular to flow. Screen aperture in the range of 250 to 1600 microns.	Pretreatment	No moving parts. Used for removal of gross and coarser particulate matter, cleaning device may required. Disposable sludge possible.

economics. Because of the blinding caused by suspended solids alone or in combination with oil and grease, most types of screens used on CSO require an automated on-line cleaning system.

Filtration can provide a much higher degree of pollutant removal than screening or sedimentation and is a principal method of treatment for potable water. As well, despite a relatively short period of use, filtration is already a well-established operation for achieving supplemental removals of suspended solids from wastewater treatment effluent.

Filtration is accomplished by passing the flow through a bed of granular material with or without the addition of chemicals. Within the filter bed, the removal of suspended solids is accomplished by a complex process involving one or more removal mechanisms such as straining, interception, impaction, sedimentation, and adsorption. The end of the filter run is reached when suspended solids in the effluent increase beyond an acceptable level, or when a limiting head loss occurs across the filter bed. The filtration phase is then terminated, and the filter backwashed to remove the material that has accumulated within the filter bed. This is usually done by reversing the flow and providing a sufficient flow velocity to fluidize the filter medium. In most wastewater treatment plant flow sheets, the backwash water is returned to the treatment process.

While effluents from wastewater treatment processes contain compressible solids, a large fraction of the solids in wet weather flows are non-compressible and should be readily filtered [5].

Studies on CSO have indicated the applicability of dual-media filtration using an anthracite layer on top of fine sand. Pretreatment using a fine screen can reduce the solids loading to the filter and the addition of chemical coagulants enhances pollutant removals. Air injection during backwashing has proven necessary.

Regulators are commonly used to divert a portion of the storm flows in combined sewers, either to prevent surcharging downstream or to limit the rate of flow to a treatment plant. Over the past few years, two regulator/concentrators have been developed for CSO: the swirl concentrator and the helical bend separator. These have the capability

of providing both quality and quantity control from induced hydraulic flow patterns that separate the solids from the main flow (see Section 5.2.8 and Figure 35).

Helical bend concentrator/regulators have been modelled and design criteria and comparative cost evaluations have been developed [49]. Although no demonstration projects have been implemented in North America, helical bends appear more practical as in-line devices than as satellite or off-line devices.

Swirl concentrators have been modelled and demonstrated for various processes including treatment and flow regulation, grit removal, primary treatment, and erosion control. A swirl concentrator is shown in Figure 49. During dry weather, all of the flow passes through the concentrate outlet (F) to the interceptor sewer. Under this condition, the swirl concentrator functions as an empty tank and provides no treatment. During storm events, the increase in flow causes the level in the concentrator to rise resulting in rotary motion of the flow and concentration of the solids. The concentrate passes to the interceptor as foul underflow. The relatively clear supernatant water passes through the flow outlet (E) and can be stored, further treated or discharged. Pumping of foul underflow can be avoided if the regulator is fitted between the hydraulic gradient of the combined sewer and the interceptor.

5.4.3.2 Physical-chemical processes. Solids separation by flotation is achieved by introducing fine air bubbles into the liquid phase. These bubbles attach themselves to the solid particles and increase their buoyance, causing them to rise. The floated particles are removed by skimming. The process is shown schematically in Figure 50. Since the flotation rise velocity is generally greater than typical particle settling velocities, higher overflow rates and lower detention times can be used in flotation than in conventional settling.

Air is normally introduced into a pressurized portion of the flow, either influent or recycled effluent. When the pressure is reduced in the flotation tank, the air comes out of solution in the form of fine bubbles. This method of bubble generation is generally preferred to the use of spargers because it produces smaller bubbles and gives a more even distribution in the tank. Performance is affected by the mode of

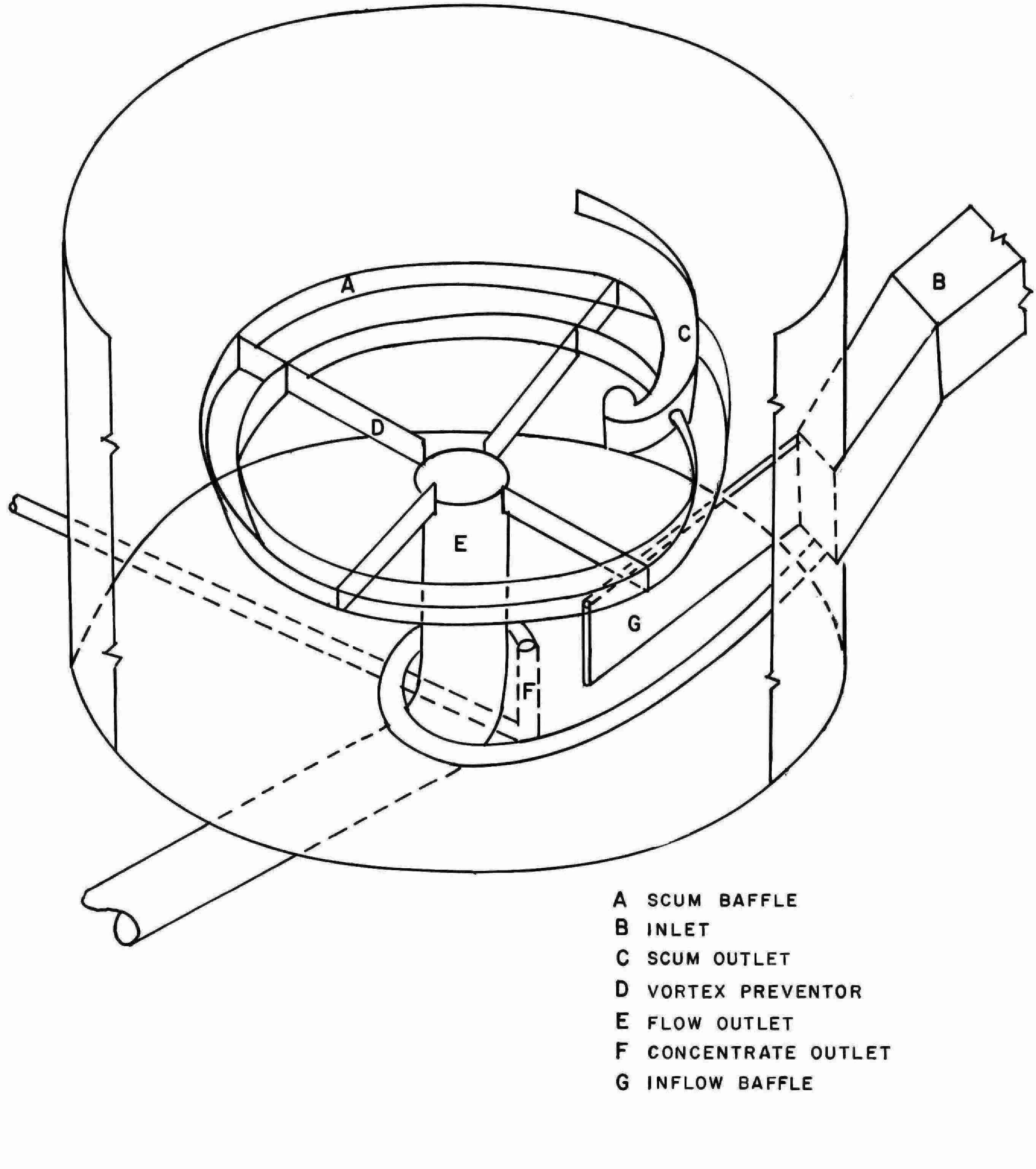


FIGURE 49. SWIRL CONCENTRATOR

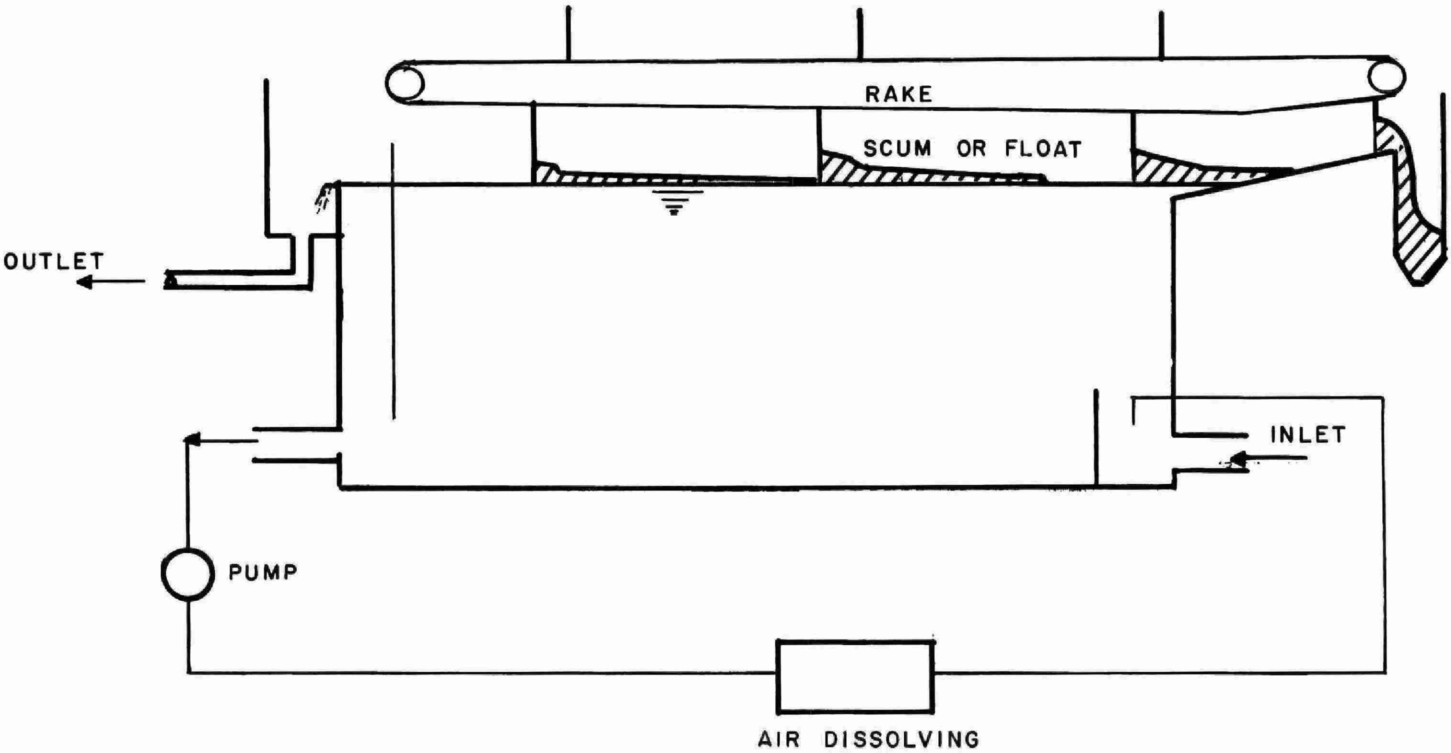


FIGURE 50. DISSOLVED AIR FLOTATION

pressurization, saturation pressure, and air to solids ratio. Performance is also dependent on the skimmer height and speed. Chemical coagulants such as ferric chloride, alum, and polymers typically are used to improve performance.

For wet weather treatment, flotation is almost always preceded by coarse screening to remove large solids. Easily settleable materials may be removed by fine screens.

5.4.3.3 Biological processes. At present, application of biological treatment to wet weather flows has been limited to the contact stabilization modification of the activated sludge process, high-rate trickling filters, rotating biological contactors, and lagoons. Removal efficienc ies are normally lower than for sanitary sewage [49]. With the possible exception of lagoons, an active biomass for these processes normally must be obtained from a dry weather WPCP. This restricts the location of wet weather biological treatment facilities and means that an acclimatization period is required before a good effluent is produced. Equalization or

storage of the wet weather flows is essential to prevent washout of the biomass. Except for lagoons, final clarification is normally required to remove the biological solids generated by the process.

The problem of obtaining or maintaining an active biomass together with high initial capital costs are considered serious drawbacks in utilizing biological systems solely for wet weather treatment [49]. Dual use has therefore been explored in some demonstration programs. Compared to physical treatment processes, most biological systems are very susceptible to overloading and shock loads.

Biological treatment is expected to be applicable only to CSO. Storm water generally has a relatively low BOD when data are flow-weighted and the need for biological treatment in the absence of "fugitive" or sanitary waste inputs is unlikely.

In the contact stabilization process (Figure 51), wastewater is mixed with returned activated sludge in an aerated contact basin for about 15-60 minutes to permit absorption of organics by the sludge floc.

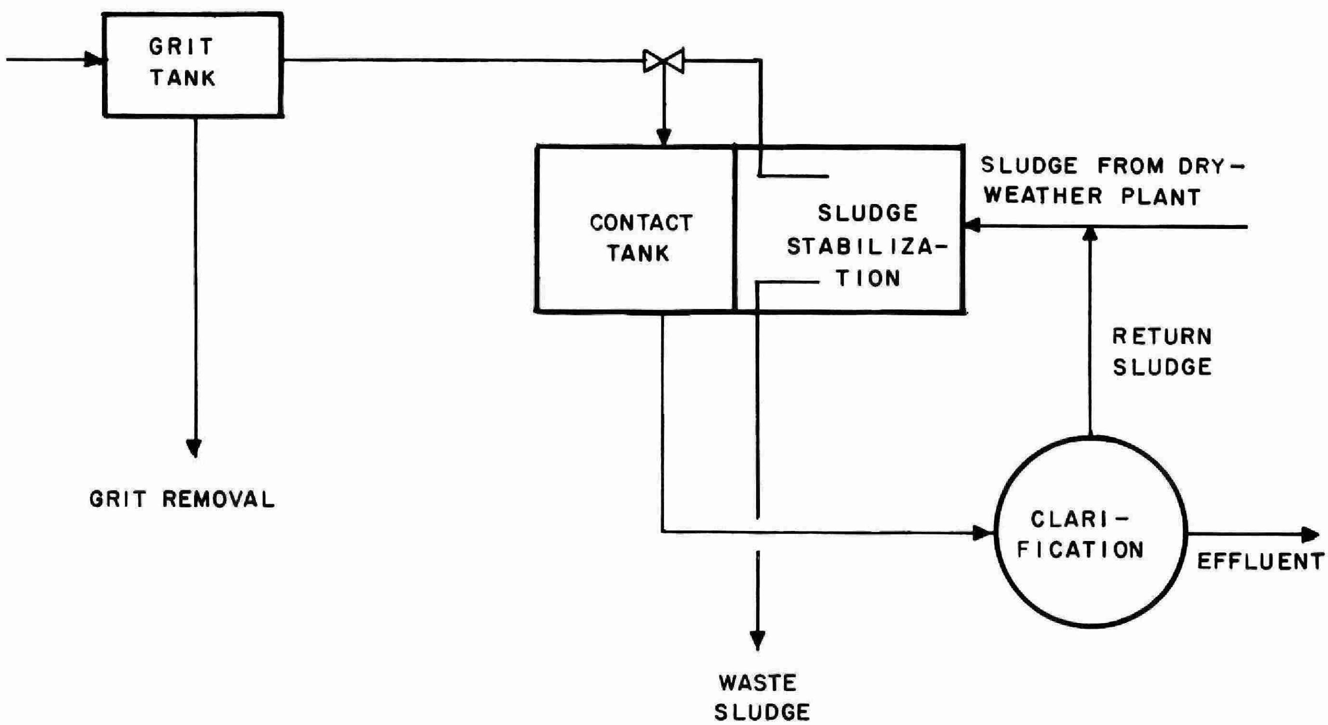


FIGURE 51. CONTACT STABILIZATION PROCESS DIAGRAM

The sludge is then separated from the effluent in a clarifier and aerated in a stabilization basin for three to six hours to permit assimilation of the absorbed organics. The stabilized sludge is returned to the contact basin as required. In Ontario, there are only a few contact stabilization plants treating sanitary waste. Many of the plants initially designed for contact stabilization are being run as extended aeration plants. Although it is doubtful that the contact stabilization process would be applied for wet weather treatment in Ontario, U.S. experience with use of the process in the treatment of CSO is discussed briefly in section 5.4.5.

A trickling filter is usually a large-diameter tank filled with crushed stone, slag, hard coal, redwood slats or corrugated plastic. Wastewater is applied either intermittently or continuously over the top of the filter by means of a rotating distributor. As the wastewater trickles down the filter, organic pollutants are oxidized by organisms attached to the filter material. Aerobic conditions are maintained by a countercurrent flow of air through the filter bed. A trickling filter is generally preceded by primary treatment and followed by settling to remove sloughed biomass from the filter effluent. Although trickling filters are widely used for the biological treatment of sanitary sewage in many parts of the world, Ontario has only three relatively old plants remaining in operation. The process has operated poorly under Ontario climatic conditions and is not likely to be favoured for dual use treatment in this province.

The rotating biological contactor (RBC) is a supported-growth wastewater treatment process which was developed and introduced on a commercial scale in Europe in the later 1950's and early 1960's. The RBC consists of a series of closely-spaced discs anchored on a horizontal shaft which is supported above the surface of the wastewater in a semi-circular or rectangular tank. The shaft rotates exposing the biological growth on the disc surfaces alternately to the wastewater and to the atmosphere. Potential advantages of RBC's are low power requirements and the ability to handle flow variation and shock loads. As with trickling filters, final clarification is normally required to permit settling of sloughed biomass.

The three basic types of lagoons are aerated lagoons, facultative lagoons, and aerobic stabilization ponds. In the aerated lagoon, aerobic conditions are maintained by mechanical aeration. In facultative lagoons oxygen is maintained in an upper aerobic layer by the presence of algae. This overlies an anaerobic bottom zone and an intermediate zone that is partly aerobic and partly anaerobic. Aerobic stabilization ponds are shallow lagoons in which algae and atmospheric diffusion maintain aerobic conditions throughout.

Lagoons have been used or suggested for use on CSO in various ways, including dual use polishing lagoons treating discharge from both a wet and a dry weather treatment facility [50]. However, in Ontario, high land costs will likely be a limiting factor on the use of lagoons in larger urban areas.

Effluent from lagoons may require additional treatment for control of algae or floatable solids. Winter operation of mechanical surface aerators can present difficulties because of icing.

5.4.3.4 Disinfection. The purpose of wastewater or storm water disinfection is to reduce the levels of pathogenic microorganisms sufficiently to protect the public from water-borne diseases. Since it is known that both storm water and CSO contain significant organic materials, regrowth of microorganisms can be expected after discharge to surface waters under favourable conditions, unless adequate treatment has also been provided.

Total coliforms, fecal coliforms, and fecal streptococci are the most common biological indicator organisms used to measure water, wastewater and storm water pathogenic quality and disinfection efficiency. When present in wastewater above critical levels, these indicators generally imply the presence of pathogenic microorganisms. The microorganisms in CSO and storm water are of varied origins. While the fecal material in surface runoff is primarily of animal origin [44], storm water may also be contaminated with sanitary sewage from illegal connections, deliberately engineered overflows or other sources. Studies are being conducted to identify other microorganisms which can serve as more precise measures of human health hazards for storm flows [49].

For water and wastewater, the most common disinfectant is chlorine. For treated domestic waste, a contact time of 15 to 45 minutes and a chlorine residual of 0.2 to 2.5 mg/L are commonly recommended.

Several aspects of conventional chlorine disinfection require reconsideration for wet weather treatment applications:

- Locations at which disinfection is required are often at outlying points on the sewer system and may require unmanned and automated facilities.
- The transport of bulk liquid chlorine to remote or unmanned sites may be judged as presenting an unacceptable hazard to the community and may be subject to restrictions.
- Fluctuations in storm flow quantity and quality severely tax the control capability of the best chlorine dosing and flow-pacing equipment. Precise control of end-of-contact residual is impractical.
- Contact times of more than 10 minutes for disinfection represent an appreciable extra "storage" cost.

In high-rate disinfection systems where contact times are usually in the range of one to five minutes, adequate mixing is critical to provide complete dispersion of the disinfectant and permit contact with the maximum number of microorganisms. Mixing can be accomplished by mechanical flash mixers at the point of disinfectant addition and at intermittent points, or by use of specially designed turbulence-inducing contact chambers. Ontario experience has indicated that, despite a marginal reduction in chlorine dosage for high-rate disinfection, the end-of contact residual was much higher than for conventional disinfection. This may necessitate consideration of dechlorination where aquatic toxity is a concern.

Several demonstration projects have focused on methods to modify chlorination practice for CSO application. Improved dosing methods and shortened contact times have been investigated. On-site hypochlorite generation is a proven technique which can eliminate the need for transport or handling of gaseous chlorine.

Alternative disinfectants such as chlorine dioxide and ozone have been evaluated at pilot-scale. Chlorine dioxide shows promise and full-scale evaluations are pending.

5.4.3.5 Solids handling and disposal. A recent EPA study [46] determined the characteristics of sludges produced by operating CSO demonstration treatment systems. Various dewatering processes were evaluated at bench scale. In addition, a desk-top analysis evaluated the pump/bleedback of wet weather sludges to a dry-weather sludge handling/treatment and disposal facility.

The study indicated that the volumes and characteristics of the residuals produced from CSO treatment vary widely. Sludge volumes were higher than for sanitary treatment, ranging from less than 1% up to 6% of the raw volume treated and containing 0.12% to 11% suspended solids. The volatile content of these sludges varied between 25% and 63%. Biological treatment sludges showed the highest volatile content and fuel values. The heavy metal and pesticide concentrations of the various sludges were significant.

For low solids content residuals (storage, screen backwash, waste activated sludge, etc.), gravity or flotation thickening were considered to be the optimum steps for further removal of the water fraction. Centrifugation and vacuum filtration were regarded as the optimum dewatering techniques for the high solids content residuals (settled storage treatment sludge, flotation scum and other thickened sludges) prior to their ultimate disposal by landfill or incineration. The fuel values of the CSO sludges indicated that a significant amount of auxiliary heat would be required to sustain combustion.

It was concluded that the pump/bleedback of CSO treatment residuals may not be practical for an entire city because of the possibility of hydraulic and/or solids overloading of the dry-weather treatment facilities and other adverse effects such as shock loads of toxic heavy metals. Controlled pump/bleedback on a selective basis may be feasible.

It was concluded that the ultimate choice of sludge handling and disposal methods for CSO would be site-specific and should include treatability tests.

Little information on sediment composition or rate of buildup in storm water detention basins is available. At the Southdale Lakes in the City of Winnipeg, a sediment buildup of 25 cm (1 in) per year has been reported [33]. The lakes have a surface area of 13 ha (32.2 acres) and service a 225 ha (555 acre) residential area. The same sediment buildup rate has been projected in the proposed Meadowvale West Lake sedimentation basin in the City of Mississauga, where a 0.4 ha (0.95 acre) sedimentation basin will receive runoff from 107 ha (265 acres) [24]. In Lake Whetstone in Montgomery Village, Maryland, an accumulation of 0.7 m (2.5 ft) of sediment was removed after 10 years of receiving storm water from a residential-agricultural area. This is an average buildup of 7.5 cm (3 in) per year.

Basins have generally required cleaning within the first year if the serviced area is under development, largely because eroded soil from construction sites has reached the storm sewer and then the detention basin. Once development has stopped and disturbed areas have been resodded, sediment buildup in the basins is substantially lower.

5.4.4 Use of Storm Water Management Model (SWMM)

The SWMM contains a section which calculates the cost and expected effluent quality from various types of wet weather treatment processes. The model is only applicable to CSO. Table 36 shows the available treatment options and Figure 52 shows the possible treatment combinations. A complete description of formulae and design assumptions is provided in the Canadian SWMM study [51]. The relation of the treatment block to the complete SWM model is discussed in Chapter 3 of this manual.

To date, the SWMM is limited to routing and treatment of BOD, TSS, and coliforms. The value of the coliform routing is questionable for practical use and the accuracy of the BOD and TSS routings is not high. Accordingly, the SWMM program should only be used to give a preliminary indication of effluent quality and relative costs of various alternatives for treatment of combined sewage.

TABLE 36. SWMM TREATMENT PROCESS OPTIONS [52]

1)	Storage/Sedimentation
2)	Screening and Filtering Units
	A. Bar racks
	B. Fine screens
	C. Microstrainers
	D. Effluent screens
	E. High-rate filters
3)	Concentrating Units
	A. Swirl concentrator
	B. Dissolved air flotation
	C. Sedimentation
4)	Biological Treatment
5)	Disinfection
	A. Contact tank
	B. High-rate disinfection

The limitations to the treatment subroutine are:

1) different processes are modelled with varying degrees of sophistication, ranging from simple assumptions to complex equations;
2) the data base for some algorithms is very limited;
3) treatment algorithms have been developed from steady-state operation;
4) biological treatment and (in the Canadian SWMM) sedimentation processes are based on sanitary waste experience; and
5) the lower limit of scale (19 000 m^3/d - 5 U.S. mgd) is large for Ontario conditions.

The general cost formula for each treatment unit is given by:

$$S = AQ^BF$$

where: S = treatment cost,

A = base cost factor,

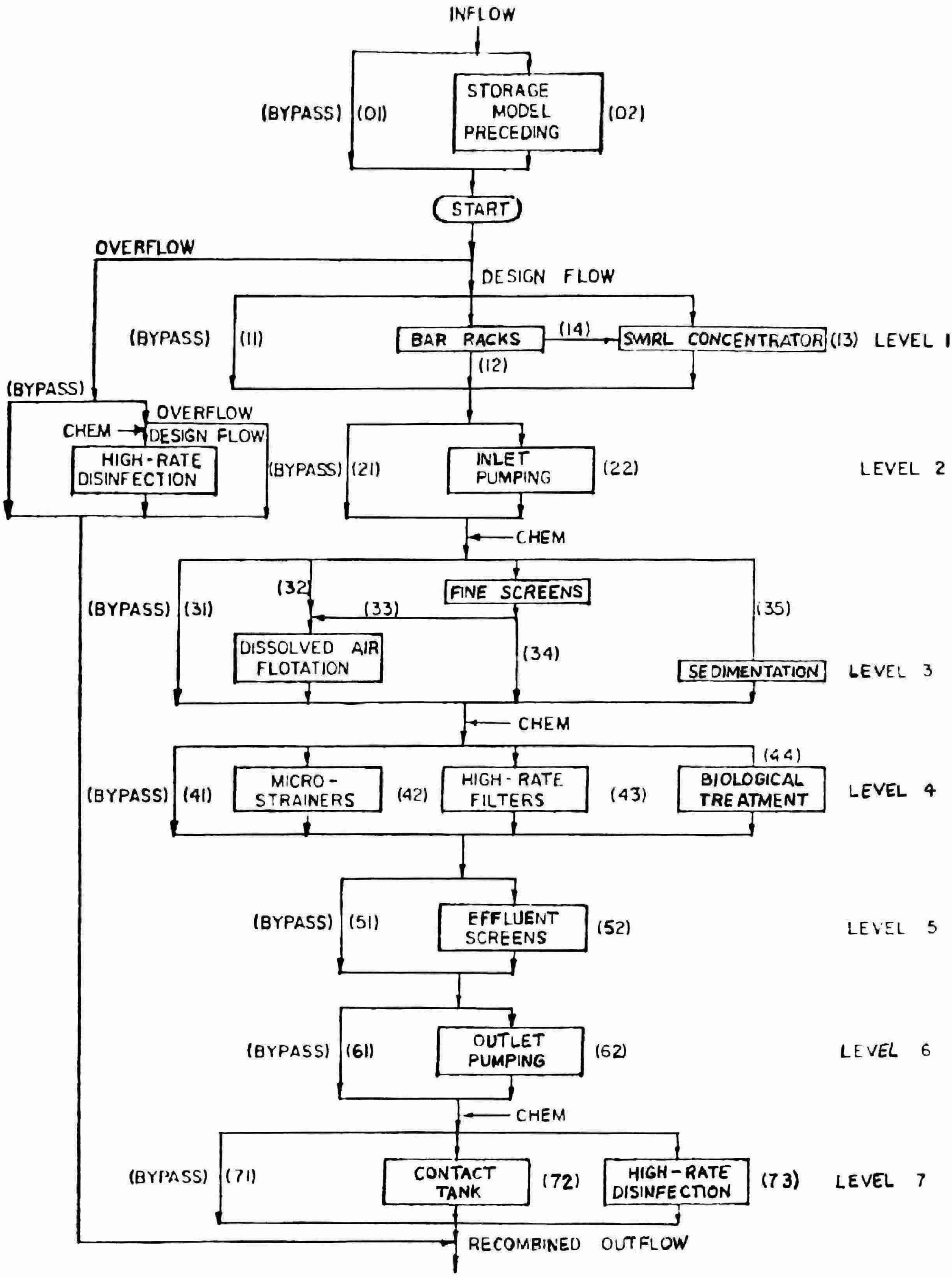


FIGURE 52. OPTIONS AVAILABLE IN SWMM TREATMENT PROGRAM [53]

Q = design flow rate,

B = economy of scale indicator,

F = specific time and locality factor.

Output includes annual costs and event costs for chemicals, operation, and maintenance. Although the cost data base for the model is limited, the SWMM represents one of the best sources of preliminary cost data presently available and covers most of the available treatment processes. Various capital cost functions adjusted for Toronto have been reported [51].

5.4.5 Application of treatment processes to combined sewer overflows

5.4.5.1 Physical processes. Removal of pollutants by sedimentation has shown erratic results for both suspended solids and BOD in combined sewer overflow applications. Suspended solids removal as a function of hydraulic loading rates is presented in Figure 53 for typical CSO sedimentation facilities. The results represent average suspended solids removals for a storm event, using average hydraulic loading rates during the overflow period. The data scatter is indicative of high and changing hydraulic loading rates and variable influent concentrations.

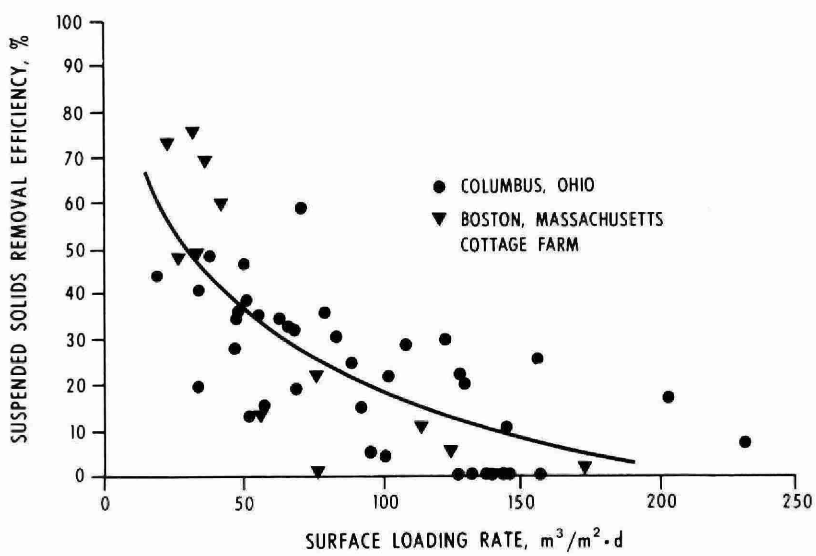


FIGURE 53. TYPICAL SUSPENDED SOLIDS REMOVAL EFFICIENCIES FOR STORAGE/ SEDIMENTATION FACILITIES WITHOUT CHEMICAL ADDITION [49]

Typically, 50-60% removal of suspended solids is achieved by conventional sanitary sewage settling tanks with surface loading rates of approximately 40 $m^3/m^2 \cdot d$ (1000 U.S. $gal/ft^2 \cdot day$). Similar removals are obtained for storm water loadings in this range; however, loading rates can vary up to six times this value with removals in the range of 0 to 35%. When removals attributed to total flow capture during small overflow events and that retained by storage/sedimentation during large events are included, removals can range 60% and higher.

Removal of BOD is more erratic and ranges from 0 to 50% for most loading rates and influent concentrations. Based on typical performance of several sedimentation facilities, average BOD removal rates in excess of 20% are common [50].

Removal of heavy metals, nitrogen, phosphorus, and other constituents by sedimentation was less than 50% on the basis of a small number of samples [50].

Suspended solids removals obtained by various screening devices are summarized in Figure 54. The results should be viewed as approximate since the data exhibited considerable scatter. Percentage removal increased with increasing influent suspended solids concentrations, particularly for the smaller screens. Although microstrainers have the highest removals, their hydraulic loading rates were the lowest at 7-30 $L/m^2 \cdot s$ (10-45 U.S. gpm/ft^2) of submerged screen. In general, the results of full-scale demonstration projects have been below expectations [50].

Mechanical and operating problems have been reported for all except static screens. Rotary screens create a concentrate which is dependent on the amount of washwater used and which requires additional treatment. Drum screens, and possibly disc screens, appear to be applicable as pretreatment devices to such processes as dissolved air flotation and high-rate filtration.

Figure 55 illustrates filtration performance as a function of solids loading for high rate, dual-media filtration demonstration projects. Suspended solids removal varied directly as the influent suspended solids concentration and inversely with the hydraulic loading rate. The addition of chemicals greatly enhanced removal of suspended solids, BOD, phosphorus, and COD. Suspended solids removal with and

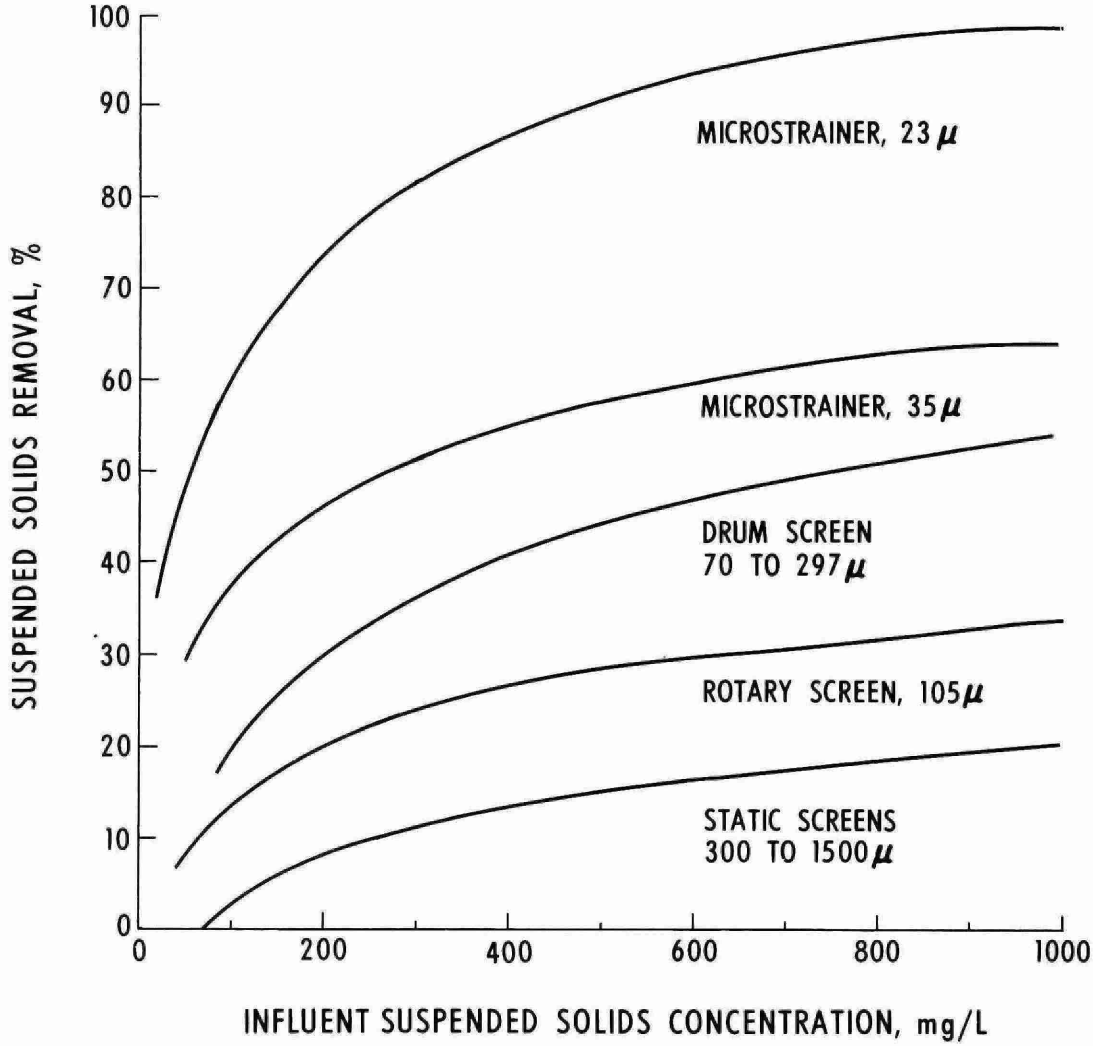


FIGURE 54. COMPARATIVE SCREEN PERFORMANCE FOR REMOVAL OF SUSPENDED SOLIDS [49]

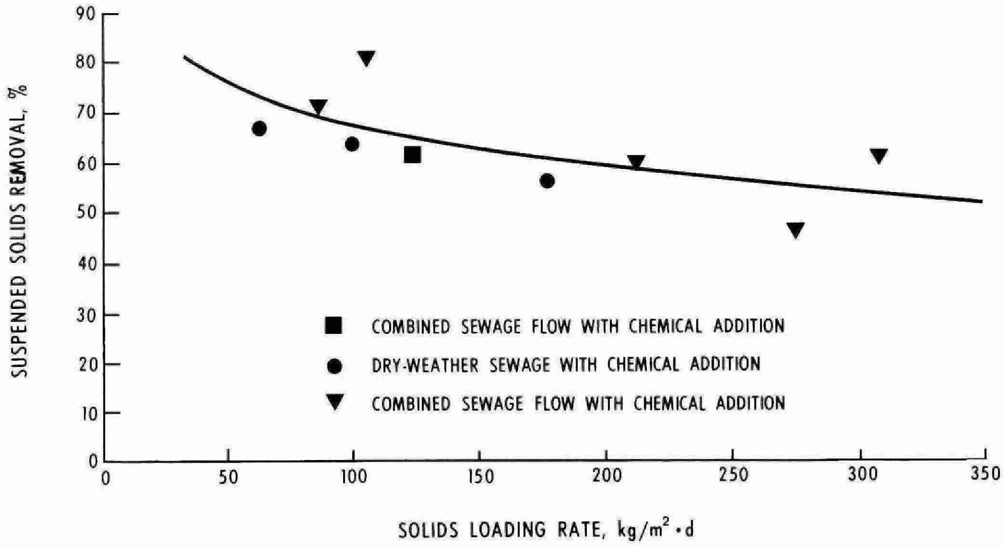


FIGURE 55. MEAN HIGH-RATE FILTRATION PERFORMANCE AS A FUNCTION OF SOLIDS LOADING RATE [49]

without polyelectrolyte addition is shown as a function of hydraulic loading in Figure 56. No correlation has been established between BOD removal and hydraulic rate because of the varying ratio of soluble to total BOD.

Swirl concentrator/regulators have shown a steady and attractive solids removal performance over a wide range of hydraulic loading rates. Units have been demonstrated up to 3.6 m (12 ft) in diameter for design flows up to 25 700 m^3/d (6.8 U.S. mgd). The swirl flow principle has also been successfully demonstrated as a grit separation device [49]. Investigations are proceeding on its potential use as a portable erosion/construction site treatment device.

Suspended solids removal of swirl concentrators has averaged approximately 50% (total mass basis) on combined sewer overflows. In addition to the removal obtained by the physical splitting of flows, as with conventional regulators, the additional 20 to 30% reduction in the suspended solids concentration is attributed to the action of the swirl. Limited data indicate a BOD mass removal of approximately 67% with a reduction of BOD concentration in the effluent of approximately 47%. These tests were conducted at flow rates substantially less than the design capacity of the swirl concentrator, and the removals obtained may be uncharacteristically high [49]. Figure 57 shows the efficiency of the swirl concentrator/flow regulator for suspended solids removal by mass and by concentration. Hydraulic loading rates to the swirl ranged from approximately 200 to 1 200 $m^3/m^2 \cdot d$ (5000 to 30 000 U.S. gal/$ft^2 \cdot d$).

Although no prototype helical bend facilities have been constructed in North America, it was found through model studies that the helical bend is capable of higher removal efficiences, with less head loss than the swirl concentrator [49]. The studies have developed design criteria and guidelines for field installations.

5.4.5.2 Physical-chemical processes. A comparison of dissolved air flotation (DAF) performance efficiency with and without the use of chemicals is presented in Figure 58. Hydraulic loading rate and influent suspended solids variables are grouped together as mass solids loading rate. Limited data were available at high mass loading rates; therefore, individual DAF run data were used instead of average grouped data to

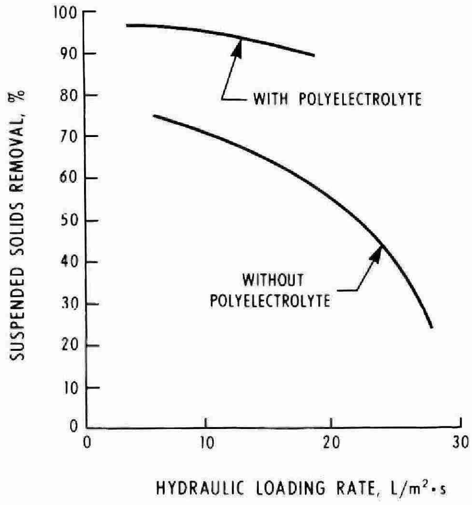


FIGURE 56. OPTIMIZED HIGH-RATE FILTRATION SUSPENDED SOLIDS REMOVAL WITH AND WITHOUT POLYELECTROLYTE ADDITION AS A FUNCTION OF HYDRAULIC LOADING RATE [49]

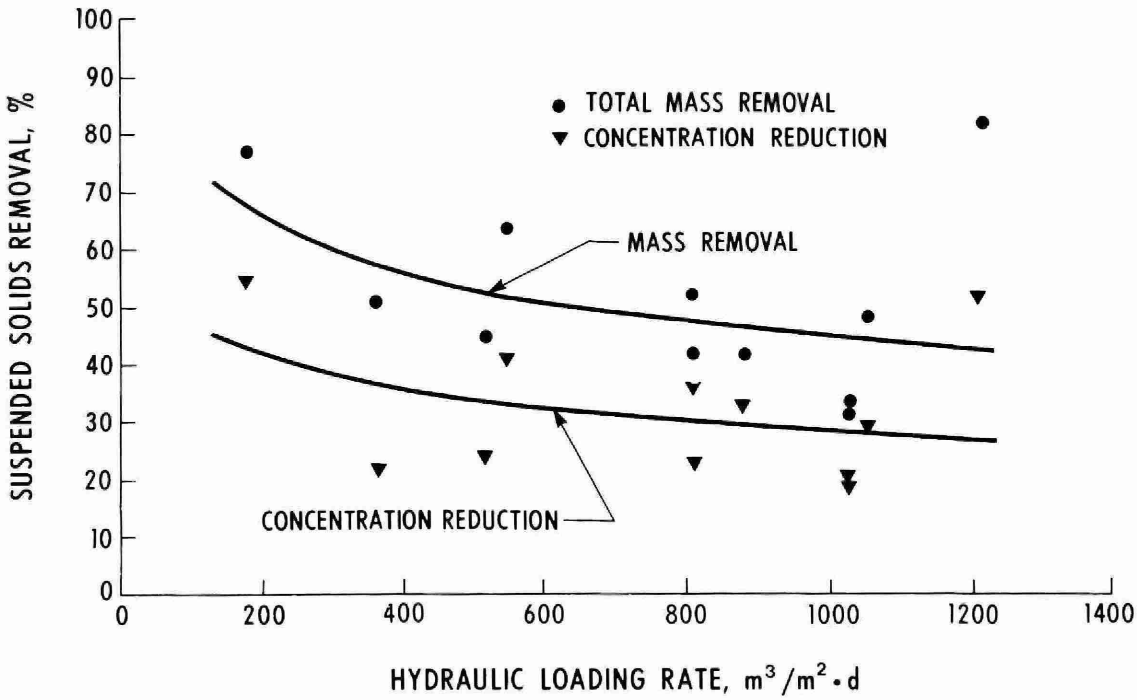


FIGURE 57. SWIRL CONCENTRATOR/FLOW REGULATOR SUSPENDED SOLIDS REMOVAL AS AS A FUNCTION OF HYDRAULIC LOADING RATE [49]

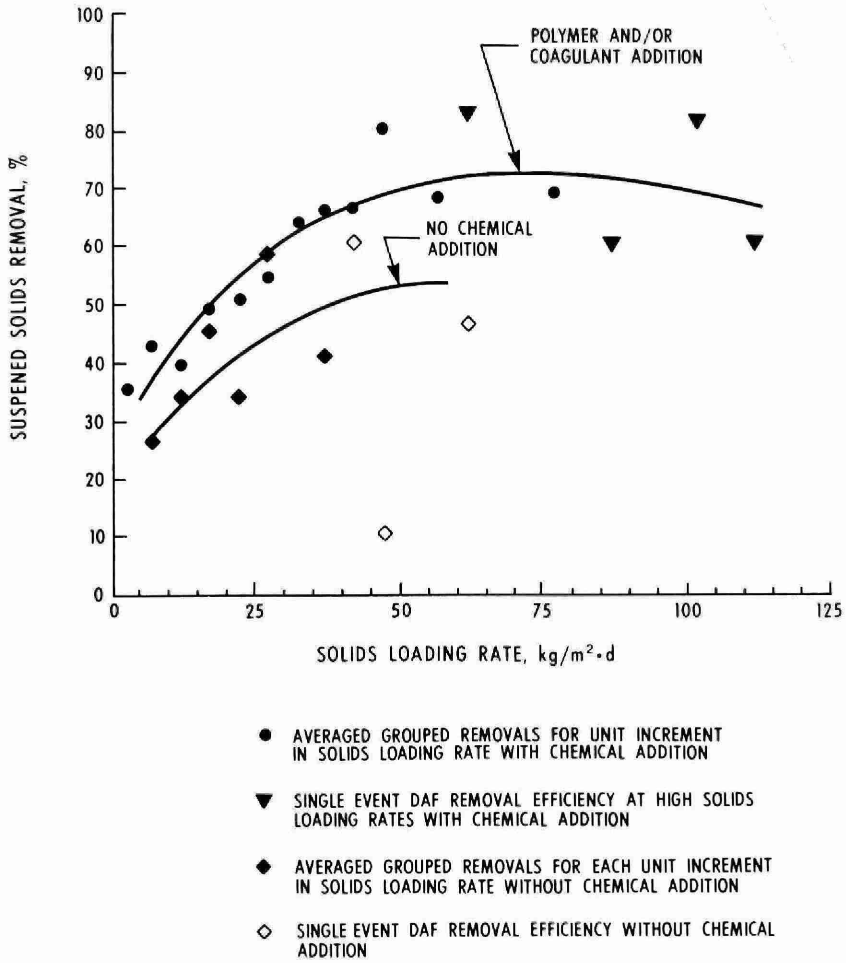


FIGURE 58. DISSOLVED AIR FLOTATION PERFORMANCE AS A FUNCTION OF SUSPENDED SOLIDS LOADING RATE WITH AND WITHOUT CHEMICAL ADDITION [49]

represent process efficiency. Data on flotation performance without chemical addition are limited since chemicals are used in most applications of this process to enhance pollutant removals.

Treatment efficiency on an overall mass basis was higher than if efficiency was calculated giving equal weight to each event without regard to volume treated (Table 37). Treatment efficiency is usually greater for the longer duration, high total volume storms than for short duration, low volume storms. This difference was attributed to the startup lag time of 30 to 45 minutes before good quality effluent was achieved [49]. Higher mass loadings and suspended solids concentrations will also affect DAF efficiency, providing a greater chance for physical contact with the float bubbles.

Dissolved air flotation requires complicated chemical feed and metering equipment to add chemicals in proportion to flow. Process efficiency will be impaired by changing flows and hydraulic overloading.

5.4.5.3 Biological processes. Biological treatment has not been demonstrated extensively for CSO. Prototype facilities include the contact stabilization process at Kenosha, Wisconsin, dual-use trickling filters at New Providence, New Jersey, biological contactors at Milwaukee, Wisconsin, and several lagoon systems. Except for the lagoons, the CSO treatment was dependent on the biomass produced from dry weather sewage plants.

Typical pollutant removals for contact stabilization, trickling filters, and RBC's are presented in Table 38, for wet-weather loading conditions. These processes include primary and final clarification. Final clarification greatly influences the overall performance of the system by preventing the carryover of biological solids produced by the processes.

The plastic media and conventional rock media trickling filters at New Providence, New Jersey operate in series during dry weather, and in parallel during wet weather. When the system is operated in the parallel mode, overall average pollutant removal is decreased and is dependent on hydraulic flow.

A demonstration scale RBC at Milwaukee, Wisconsin, confirmed previous pilot plant results by handling a high range of organic and

TABLE 37. COMPARISON OF POLLUTANT REMOVALS ON AN ARITHMETIC MEAN AND MASS BASIS FOR DISSOLVED AIR FLOTATION FACILITIES AT RACINE, WISCONSIN [49]

Site	Parameter	Percent Removed: Per event	Percent Removed: Overall Mass
I	BOD	50.1	62.4
	Total organic carbon	47.1	60.0
	Total solids	25.7	28.1
	Suspended solids	59.7	67.6
	Volatile suspended solids	64.7	73.6
	Total phosphorus	46.6	53.2
II	BOD	60.4	69.5
	Total organic carbon	50.4	66.6
	Total solids	37.6	47.2
	Suspended solids	66.1	69.8
	Volatile suspended solids	57.0	67.3
	Total phosphorus	60.3	62.4

TABLE 38. TYPICAL WET-WEATHER BOD AND SUSPENDED SOLIDS REMOVALS FOR BIOLOGICAL TREATMENT PROCESSES [49]

Biological treatment process	Expected range of pollutant removal, %*: BOD	Expected range of pollutant removal, %*: Suspended Solids
Contact stabilization	70 - 90	75 - 95
Trickling filters	65 - 85	65 - 85
Rotating biological contactors**	40 - 80	40 - 80

* Assuming primary and final clarification are provided.
**Removal reflects flow ranges from 30 to 10 times dry weather flow.

hydraulic loads for periods of 8 to 10 hours [49]. A comparison of organic removal efficiencies for the pilot plant studies (using raw sewage) and the full-scale wet weather demonstration facilities is shown in Figure 59. As hydraulic residence times fell below about 8 to 10 minutes, the organic removal efficiency of the demonstration facility dropped significantly. This treatment system was installed as an in-line device without final clarification. Final clarifiers could greatly increase BOD and suspended solids removal by removing sloughed biomass.

Pollutant removal efficiencies at treatment lagoons have varied from 85 to 95% to negative values due to excessive algae production and carry over. In addition to the type of lagoon and the number of cells, major factors that have been found to influence removal efficiencies include detention time, source of oxygen supply, mixing, organic and hydraulic loading rates, and algae removal mechanisms.

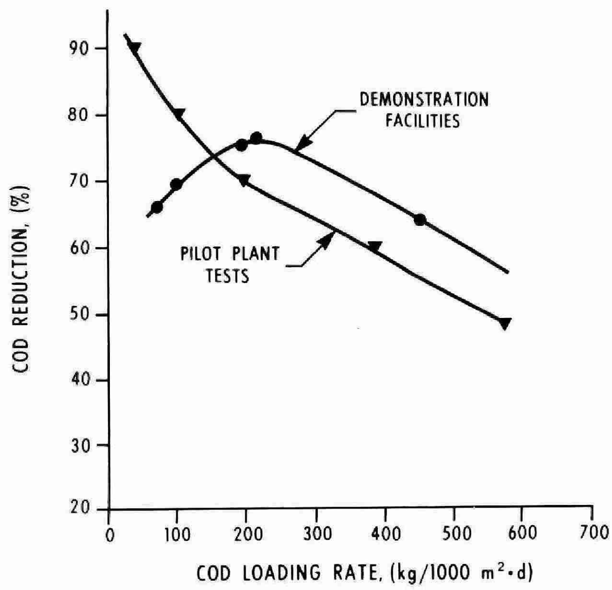


FIGURE 59. COMPARISON OF COD REMOVAL PERFORMANCE FOR PILOT AND FULL SCALE DEMONSTRATION RBC FACILITIES, MILWAUKEE, WISCONSIN [49]

A comprehensive wet weather demonstration program at Mount Clemens, Michigan incorporated a three-stage lagoon system with micro-straining for suspended solids removal between lagoons, and pressure sand filtration for suspended solids, BOD and algae control prior to final discharge. Mechanical surface aeration was provided in two lagoons while the third was an aerobic stabilization pond. This facility produced a final effluent with flow-weighted average BOD and SS concentrations of 6 mg/L and 14 mg/L, respectively.

In general, maintenance problems experienced by wet weather biological facilities are similar to those experienced at conventional biological installations. Winter operation of mechanical surface aerators has had some serious drawbacks, including icing, tipping, and sinking.

5.4.5.4 Disinfection. Some demonstration projects evaluating CSO disinfection technology are summarized in Table 39. In United States installations, as in Canada, chlorine disinfection in various sedimentation-disinfection facilities has typically been actuated simultaneously with the beginning of inflow to the basin. The validity of this approach for design is doubtful because contact time and residual can both vary widely.

5.4.5.5 Solids handling and disposal. Various means are used to remove deposits from storage/sedimentation tanks and basins. Only bench scale dewatering data for CSO sludges are available. To date sludges from full-scale CSO treatment installations have either been returned to the combined sewer for handling at the dry weather WPCP, directly landfilled, or in the case of dual-use installations, handled at the dry weather WPCP sludge facilities.

Equipment used for the collection and removal of solids from storage/sedimentation basins include rakes, chain and flights, water jets, and mechanical mixers. Rakes are the most expensive and are susceptible to failures because of long periods of inactivity or nonsubmergence. High pressure water jets are a favoured solution in many designs. Supplemental manually-operated fire hoses have proven useful as standby equipment.

The Humboldt Avenue detention tank in Milwaukee, Wisconsin uses seven mechanical mixers to resuspend settled solids after a storm for

TABLE 39. SUMMARY OF DEMONSTRATION DISINFECTION PROJECTS [49]

Project Location	Disinfecting Agent	Source	Description of Disinfection System	Period of Operation
Boston, Massachusetts Cottage Farm detention and chlorination station	Sodium hypochlorite (NaOCl)	Purchased/ stored	Automatic disinfection system injects up to 11 000 L of 10 to 15% NaOCl into the influent channel to the detention basins for the design storm.	1971 to present
New York City, New York, Spring Creek	Sodium hypochlorite (NaOCl)	Purchased/ stored	Automatic disinfection system injects up to 27 200 kg/d of 5% NaOCl into the inlet sewer of the storage/detention facilities.	1972 to present
Rochester, New York	Chlorine (Cl_2) Chlorine dioxide (ClO_2)	Purchased On-site generation	Sequential addition of Cl_2 and ClO_2 with flash mixing at each point of application. Disinfection is final treatment step following sedimentation, storage, dual media filtration, and carbon column pilot facilities.	1975-1976
Syracuse, New York	Chlorine gas (Cl_2) Chlorine dioxide (ClO_2)	Purchased On-site generation	Evaluation of individual and sequential addition of Cl_2 and ClO_2 following treatment of combined sewer overflows by screening and swirl concentration.	1974 to present

pumping back to the combined sewer. No manual cleaning has yet been necessary [5].

The Chippewa Falls, Wisconsin detention tank drains back to the combined sewer. Sludge on the tank bottom is allowed to dewater and is removed by front end loader or street sweeper for transport to a sanitary landfill [5].

In Halifax, the detention tank is drained back to the sewer system after a storm. The remaining scum and settled solids are then flushed to the drain using a fire hose. At the Borough of York's Hyde Avenue detention tank, light deposits are washed to the drain with fire hoses after every storm. Heavy deposits are scraped into the drain trough by a tractor [54].

5.4.5.6 Development of process flowsheets. The individual unit processes discussed in previous sections would normally be combined to produce a treatment system. Specific examples of these are presented in the case studies in Section 5.4.8.

The sequence of processes would normally follow that shown for options in the SWM model in Figure 52. The number of processes and extent of treatment would depend on local requirements. Coarse screens or concentrator/regulators would normally be first, followed by physical or physical-chemical treatment processes such as fine screens, sedimentation or dissolved air flotation. If required, further treatment could be provided by microstrainers, high-rate filters or biological processes. Disinfection would normally be incorporated into the process sequence. Facilities would also be required to handle and dispose of sludge if it was not returned to the dry weather WPCP.

5.4.6 Application of treatment processes to storm water

There is little information available on storm water treatment and documented Ontario experience is minimal. At the present time, there is no generalized approach available and each situation must be examined on its own merits.

The bulk of suspended matter in storm water is inorganic; experience in erosion control suggests that this material can readily be removed by physical or physical-chemical processes. Control of suspended solids will give varying degrees of control for other parameters.

In new developments, natural engineering techniques can minmize erosion and sediment transport to reduce or eliminate the need for treatment.

With so little data available on storm water treatment, generalized statements are inappropriate and the results of individual studies are reported.

5.4.6.1 <u>Physical processes</u>. The relatively wide range of settling velocities observed in storm water [42] (0.15 - 100 mm/s or 0.006 - 4.0 in/sec) indicates that the effectiveness of solids separation will be dependent on the type of particulates present. Experimental settling column results have indicated that while normal sedimentation can often remove much of the particulate matter, chemical addition may be needed in some cases to augment removals or to remove fine silt and clay.

Table 40 summarizes results of natural sedimentation tests on storm water. Removals were generally greater for suspended solids than for other parameters. Settling times greater than 12 hours were required to obtain suspended solids removals of more than 90 percent.

TABLE 40. RESULTS OF UNAIDED STORM WATER SETTLING TESTS

Location	Scale	Settling Time (h)	Removals (%)					
			SS	VSS	BOD	COD	Organic N	Total P
Cincinnati, Ohio [55]	Bench	2.5	65	55	30	30	45	25
Nova Scotia [56]	Bench	1.0	77	70	--	--	--	--
Barrington, East York [57]	Bench/ Small pilot	26	75-95	--	0-70	25-80	0	0-85
Barrhaven, Ottawa [58]	Full	$\geq$12	90	--				20-50

According to a recent study [59], snowmelt runoff may have different characteristics than storm water runoff, and may not readily respond to natural sedimentation as a means of pollutant removal.

Chemically-assisted settling may be necessary if clays or phosphates are to be removed. Ketchum and Weber [60] found that lime was a useful settling agent for storm water, which commonly has a lower alkalinity than most other wastewaters. Addition of 2-4 mg/L activated silica (as SiO_2) reduced lime requirements by up to a factor of 10 when the alkalinity was about 50 mg/L as $CaCO_3$. At high alkalinity, no reduction in lime requirement was obtained by using activated silica.

No data were available on full-scale chemically-assisted settling for removal of clays or phosphorus.

The results of chemically-aided settling tests on storm water are summarized in Table 41. Suspended solids removals in excess of 90 percent were obtained in two hours settling. Phosphorus removal was improved over the unaided case at similar settling times.

TABLE 41. RESULTS OF CHEMICALLY AIDED STORM WATER SETTLING TESTS

Location	Scale	Settling Time (h)	Alum Dose (mg/L)	Removals (%) SS	Removals (%) Total P
Barrington, East York [57]	Small pilot	2	30	95	70
Barrhaven, Ottawa [58]	Bench	-	110-210	90	-
North Carolina [61]	-	-	60	97	-

Batch treatment of sewage lagoons with alum, ferric chloride or lime before spring or fall drawdown has produced effluent total phosphorus concentrations of less than 1.0 mg/L and indicator bacteria reductions of more than 90% [62]. Liquid chemical spreading was effectively carried out by adding the chemicals into an outboard motor propeller's turbulent zone. This approach may be applicable to storm water treatment.

Where storm water storage/sedimentation ponds are to serve a large area or will receive significant sediment loads, an upstream screening or settling area ahead of the main basin is desirable to protect the ponds

for multiple use. Facilities for the capture of oil or similar 'floatable' liquids may also be needed in the upstream settling area.

In sizing basins for pollution control, it has been suggested [55] that ponds retain 75% of the runoff from a one-year return storm for at least $1\frac{1}{2}$ hours. This is based on the decreased rate of removal of pollutants at longer detention times and an assumed improvement in runoff quality with time for storms of long duration. There is little additional cost in providing sedimentation facilities in ponds designed for flow control.

5.4.6.2 Disinfection. Detailed bacteriological studies on runoff in several catchments have shown that bacteria and viruses are discharged throughout a storm event. Thus there is no rationale for disinfection of the first flush only.

Disinfection of chemically-settled storm water with 2 to 6 mg/L chlorine and a 20-minute contact time produced a 99.99% kill of total coliforms, fecal coliforms and fecal streptococci [55]. Despite a chlorine residual at the end of the contact period, dechlorination resulted in regrowth of coliforms in 24 to 72 hours. There was no significant regrowth of fecal coliforms or fecal streptococci.

Results of small-scale batch treatment disinfection studies on runoff at Barrington Avenue, East York [57] showed that alum doses in the range of 20 to 50 mg/L achieved moderate removals of bacteria. Sodium hypochlorite or alum/NaOCl treatment consistently produced effective disinfection. With a 30-minute contact time, a NaOCl dose of 5 mg/L as available chlorine was sufficient for adequate disinfection in 12 of 14 events studied.

Storm water disinfection was studied at full-scale in New Orleans, Louisiana [5]. Although the city has a completely separated sewer system, frequent precipitation and a high water table produce coliform counts as high as 10×10^7 per 100 mL in storm water [63]. Disinfection was carried out at three pumping stations which pump 5.6×10^5 m^3 (20×10^6 ft^3) of storm water on rainy days. With sodium hypochlorite, greater than 99.99% reduction of coliforms was achieved with a residual of 0.5 mg/L as chlorine. Typical contact times were one to three minutes at dosages of 5 to 15 mg/L chlorine. Aftergrowth of organisms was observed.

Although storm water disinfection appears practicable, it may be insufficient in itself to guarantee desired levels of water quality indicator organisms. The need for pretreatment and the implications and likelihood of regrowth should be examined carefully in each case. If disinfection with chlorine is practised, dechlorination may be necessary to protect aquatic life in receiving waters.

5.4.7 Costs

5.4.7.1 Representative system costs. This section presents construction costs and average operating and maintenance costs for wet weather treatment processes as a guide for planners to determine the relative cost-effectiveness of various treatment alternatives on a preliminary basis. When preparing estimates for specific application or final selection of alternatives, detailed cost studies are required to account for variations in local conditions or changing design requirements.

All costs indices used in the construction industry are based on the costs of specific construction material and labour, proportioned in a predetermined manner. The most frequently used index is probably the Engineering News Record's (ENR) Construction Cost Index. However, this index does not include mechanical equipment or pipes and valves which are normally associated with water utility plant construction, and the proportion of materials and labour is not specific to water utility construction.

Although more specific indices for sewage treatment have been developed, the summary cost data presented in this section for wet weather treatment facilities are based on the ENR index. While these data are useful for estimating preliminary costs, the limitations of the indices should be borne in mind. The U.S. EPA has published a manual [26] which provides a basis for preparing detailed cost estimates of various unit processes for CSO treatment. Because of the limited Canadian experience in CSO treatment, the costs presented here are for U.S. facilities.

The representative costs presented utilize actual construction bid tabulations and estimates from storm water facilities together with data used to develop detailed cost curves for the EPA manual. All costs are adjusted to the ENR 2000 cost index. The costs of land, engineering, and contingencies are not included. In most cases, the data base from

which costs have been derived is limited since comparatively few full-scale installations exist.

A general comparison of the cost of the various physical treatment processes is presented in Table 42. The ranges of costs were estimated, and in some cases, adjusted to a plant capacity of 9500 m^3/d (25 mgd). Average capacity costs reflect an approximate cost for a treatment process, indicating relative differences in magnitude between processes.

Costs of sedimentation facilities are summarized in Table 43, with flow capacities based on a theoretical 30-minute detention time to provide an equal basis of comparison. Actual detention times based on maximum flow rates ranged from eight minutes to more than one hour.

Construction costs for swirl concentrator/regulators are presented in Figure 60 for swirl chamber diameters of 3-15 m (10-50 ft). These cost are based on estimates and actual construction costs, excluding by-pass sewers. Operation and maintenance costs have been developed based on the number of overflow events per year, and on an annual manhour basis. Actual operation and maintenance costs have been reported at approximately \$2000 per year (ENR 2000) for the West Newell Street installation at Syracuse, New York.

Costs of drum screens and microstrainers, rotary screens, and static screens are summarized in Table 44, based on estimates from actual demonstration scale facilities. For several installations, costs were also estimated for various levels of capacity based on the configuration of the demonstration installation. Capital construction costs for all screening alternatives ranged from \$3250-\$6870/1000 $m^3 \cdot d$ (\$12 300 - \$26 000/mgd) and average approximately \$5000/1000 $m^3 \cdot d$ (\$19 000/mgd). The range of capital costs reflects special construction methods, type of building, and/or support facilities required at specific sites. Operation and maintenance costs average approximately \$0.13/m^3 (\$0.50/1000 gal) and range from approximately \$0.01-\$0.03/m^3 (\$0.02-\$0.10/1000 gal) for all types of screens.

Costs of dissolved air flotation facilities used for storm water treatment have varied widely, from approximately \$5280-\$6870/1000 $m^3 \cdot d$ (\$20 000-\$26 000/mgd) to over \$18 500/1000 $m^3 \cdot d$ (\$70 000/mgd). These

TABLE 42. SUMMARY OF AVERAGE CONSTRUCTION COSTS FOR 94 600 m^3/d (25 mgd) PHYSICAL TREATMENT FACILITIES[a] [49]

Physical Treatment Process	Construction Costs ($)	Average cost ($\$/m^3 \cdot d$)	Average Cost ($/mgd)
Sedimentation[b]	238 000-850 000	60	23 000
Swirl concentrator/ regulator[c]	50 000-65 000	1.20[d]	4 500[d]
Screening[e]	400 000-600 000	5.00	19 000
Dissolved air flotation[f]	600 000-1 200 000	9.00	34 000
High-rate filtration	1 400 000-1 700 000	15.30[g]	58 000[g]

[a] ENR 2000.

[b] Adjusted to 94 600 m^3/d (25 mgd) costs.

[c] Range for 90 and 100% grit removal.

[d] Based on a 45 400 m^3/d (12 mgd) facility.

[e] Estimates include supplemented pumping where used.

[f] Based on hydraulic loading rate of 235 $m^3/m^2 \cdot d$ (5760 $gal/ft^2 \cdot d$); includes processing and chemical addition facilities.

[g] Based on hydraulic loading rate of 16 $L/m^2 \cdot s$ (24 $gal/ft^2 \cdot min$); includes prescreening and chemical addition facilities.

TABLE 43. SUMMARY OF COSTS FOR TYPICAL SEDIMENTATION FACILITIES[a] [49]

Project location	Flow capacity[b] 1000 m^3	(mgd)	Construction cost $/1000 m^3	($/mgd)	$/ha	Cost ($/acre)	Annual operation and maintenance cost $/1000 m^3	($/mgd)
Boston, Massachusetts								
Cottage Farm	236	(62.4)	27 480	(104 000)	1 040	(420)	340	(1 280)
Charles River	218	(57.6)	43 520	(164 700)	7 810	(3 160)	446	(1 690)
Columbus, Ohio								
Whittier Street	681	(180.0)	8 980	(34 000)	520	(210)	---	---
Dallas, Texas								
Bachman Storm Water Plant	218	(57.6)	8 430	(31 900)	---	---	190	(720)
Milwaukee, Wisconsin								
Humboldt Avenue	708	(187.0)	2 510	(9 500)	7 660	(3 100)	70	(270)
New York City, New York								
Spring Creek[c]	2252	(595.0)	5 300	(20 060)	9 045	(3 660)	45	(170)
Saginaw, Michigan	636	(168.0)	5 220	(19 760)	5 040	(2 040)	53	(200)

a ENR = 2000.

b Based on 30-minute detention time.

c Neglecting 49 200 m^3 (13 mil gal) of trunk sewer storage.

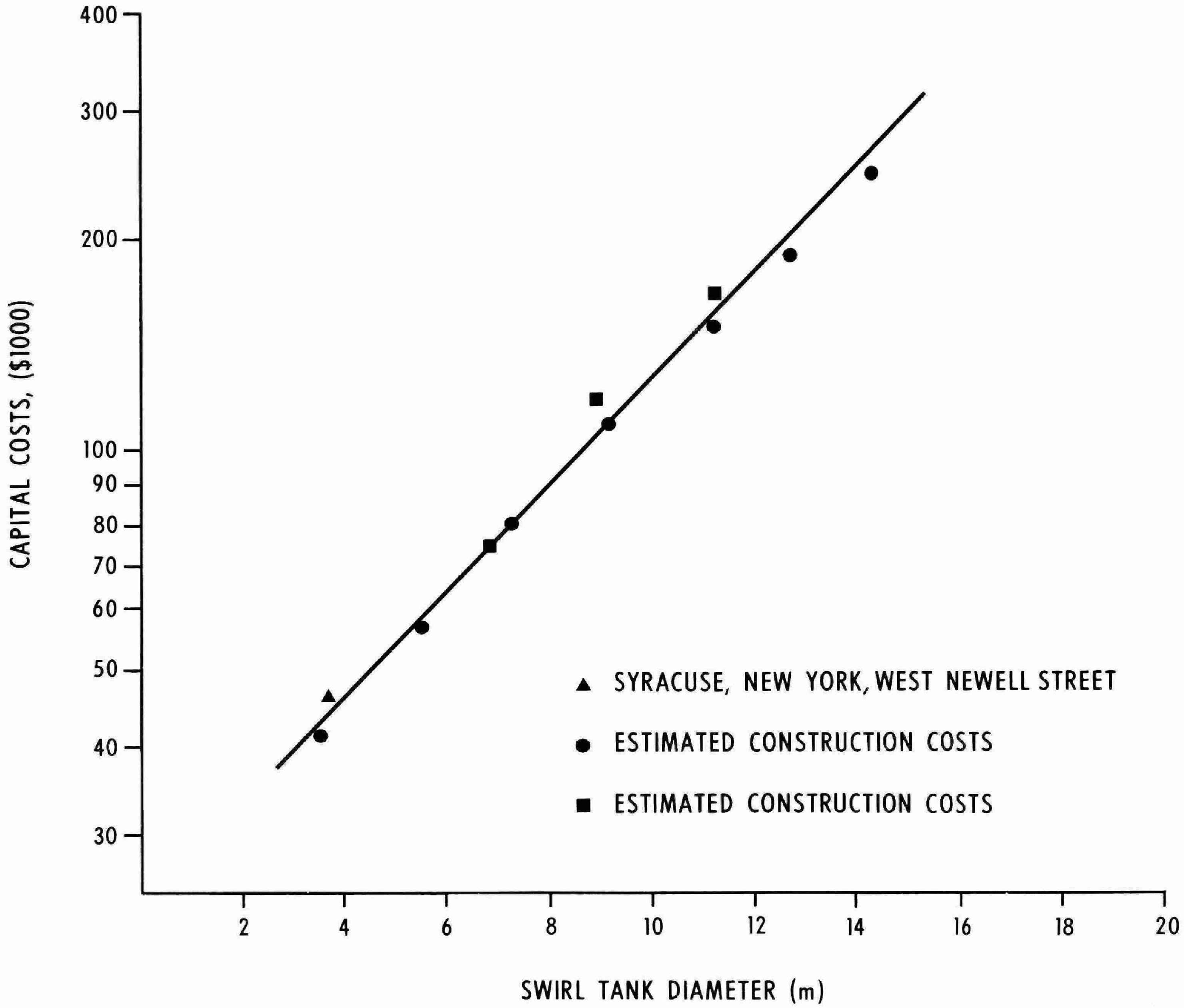


FIGURE 60. ESTIMATED CONSTRUCTION COST FOR SWIRL CONCENTRATOR/REGULATORS (ENR 2000) [49]

TABLE 44. COST SUMMARY FOR SELECTED SCREENING ALTERNATIVES[a] [49]

Project location	Type of screen	Screening Capacity 1000 m^3/d	(mgd)	Capital cost, $	Cost $/1000 $m^3 \cdot d$	($/mgd)	Annual operation and maintenance cost $/$m^3$	($/1000 gal)
Belleville, Ontario[a]	Rotary screen	6.8	1.8	33 500	4 930	18 600	0.022	0.083
		20.4	5.4	97 700	4 790	17 900	0.022	0.083
		27.3	7.2	128 400	4 703	17 800	0.022	0.083
	Static screen	2.8	0.75	14 900	5 320	19 900	0.011	0.042
		20.1	5.3	95 600	4 756	18 200	0.011	0.042
		28.4	7.5	130 700	4 602	17 400	0.011	0.042
Cleveland, Ohio[b,c]	Drum screen	94.6	25	608 500	6 430	24 340	--	--
		189.3	50	887 800	4 690	17 750	--	--
		378.5	100	1 745 200	4 610	17 450	--	--
		757	200	3 340 300	4 612	16 700	--	--
Ft. Wayne, Indiana	Static screen	68.1	18	272 400	4 000	15 100	0.005	0.020
	Drum screen	68.1	18	254 300	3 735	14 100	0.010	0.039
	Rotary screen	143.8	38	584 700	4 066	15 400	0.012	0.046
Mt. Clemens, Michigan	Microstrainer	3.8	1.0	26 200	6 895	26 200	--	--
Philadelphia, Pennsylvania	Microstrainer with chemical addition	28	7.4	90 800	3 243	12 270	0.013	0.048
	Microstrainer without chemical addition	28	7.4	147 900	5 280	19 980	0.013	0.049
Racine, Wisconsin	Drum screen	147.6	3.9	22 600	153	5 800	--	--
Seattle, Washington[b]	Rotary screen	94.6	25	600 000	6 340	24 000	0.026	0.098
Syracuse, New York[c]	Rotary screen	18.9	5	129 500	6 850	25 900	--	--
	Drum screen	37.8	10	275 000	6 800	25 700	--	--

[a] ENR = 2000.

[b] Estimated costs for several sizes of facilities.

[c] Estimates include supplemental pumping stations and appurtenances.

differences can be attributed to special structural and architectural requirements, pretreatment requirements, and more importantly, to the design hydraulic loading rate. For this reason, costs for dissolved air flotation facilities are presented as a function of tank surface area in Figure 61. The cost curves are based on the cost of several different sizes of demonstration facilities and data from dissolved air flotation facilities used in conventional solids thickening applications. These costs should be considered as a preliminary guide. Operation and maintenance costs have ranged from approximately \$0.01 to \$0.06/m^3 (\$0.05-\$0.22/1000 gal) including pretreatment.

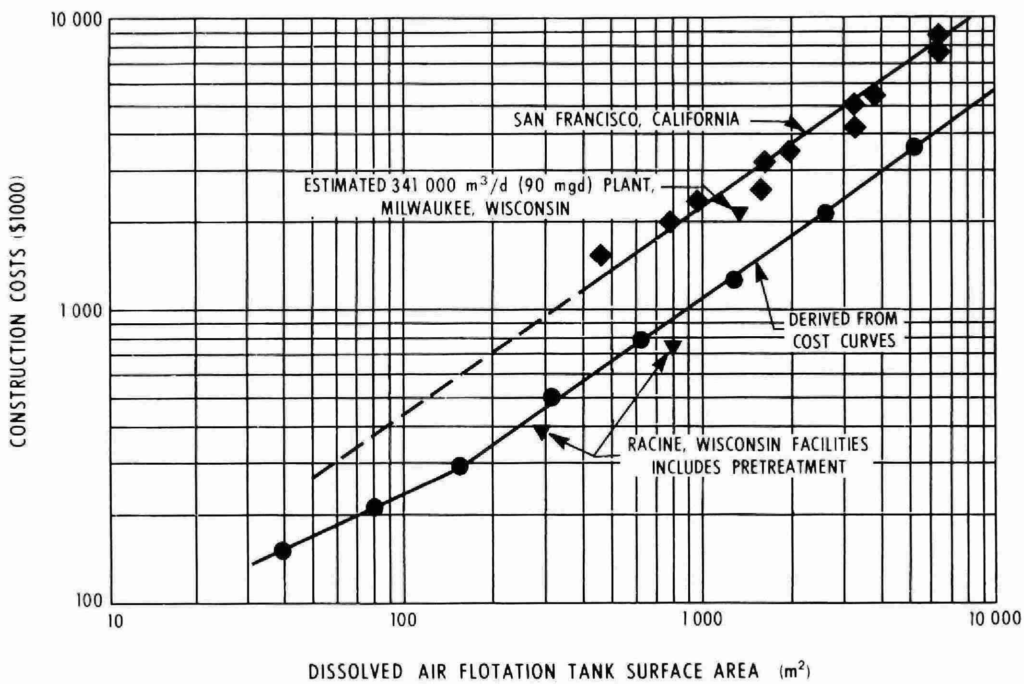


FIGURE 61. COST OF DISSOLVED AIR FLOTATION FACILITIES [49]

Cost of high-rate filtration facilities are summarized in Table 45. These costs are based on facilities of similar design to the Cleveland demonstration project and include a low lift pumping station, pretreatment by 420-micron drum screens, and chemical addition facilities. Operation and maintenance costs are based on 300 hours of operation per year.

A comparison of construction, operation, and maintenance costs for biological treatment systems is presented in Table 46. Costs of final clarification are included where retention of solids and sludge is required.

TABLE 45. SUMMARY OF COSTS FOR DUAL-MEDIA HIGH-RATE FILTRATION FACILITIES[a] [49]

Plant capacity		Construction costs,b($)		Construction costs ($/mgd)		Operation and maintenance costs, $	
m^3/d	mgd	24 gpm/ft^2 (16 $L/m^2 \cdot s$)	16 gpm/ft^2 (11 $L/m^2 \cdot s$)	24 gpm/ft^2 (16 $L/m^2 \cdot s$)	16 gpm/ft^2 (11 $L/m^2 \cdot s$)	24 gpm/ft^2 (16 $L/m^2 \cdot s$)	16 gpm/ft^2 (11 $L/m^2 \cdot s$)
94 625	25	1 440 000	1 680 000	57 600	67 200	44 000	45 000
189 250	50	2 170 000	2 620 000	43 400	52 400	55 000	57 000
378 500	100	3 980 000	4 860 000	39 800	48 600	98 000	102 000
757 900	200	6 760 000	8 020 000	33 800	40 100	129 000	134 000

[a] ENR = 2000

[b] Includes low lift pumping station, prescreening, and chemical addition facilities; and excludes engineering and administration.

TABLE 46. SUMMARY OF CAPITAL AND OPERATION AND MAINTENANCE COST FOR BIOLOGICAL TREATMENT ALTERNATIVES[a] [49]

Project location	Type of biological treatment	Peak plant capacity mgd	(1000 m^3/d)	Construction cost, $	Cost/capacity, $/mgd	($/$m^3$·d)	Cost tributary area, $/acre	($/ha)	Annual operation and maintenance cost, ¢/1000 gal (except as noted)	(¢/m^3)
Kenosha, Wisconsin	Contact stabilization	20	(75.7)	1 364 000	68 200	(18.00)	1 140	(2 820)	13.8	(3.7)
Milwaukee, Wisconsin[b]	Rotating biological contactor	4.3	(16.3)	299 000	69 200	(18.30)	8 540	(21 100)	4.4	(1.2)
Mount Clemens, Michigan										
Demonstration system	Aerated treatment lagoons	64	(242)	642 700	10 000	(2.64)	3 030	(7 490)	20.0	(5.3)
Citywide system	Storage/aerated treatment lagoons	260	(984)	5 737 000	22 000	(5.80)	3 900	(9 640)	19.0	(5.0)
New Providence, New Jersey[c]	High-rate trickling filter	6	(22.7)	475 000	79 150	(20.90)	---		12.3	(3.3)
Shelbyville, Illinois										
Southeast site	Oxidation lagoon	28	(106)	43 400	1 550	(0.41)	1 000	(2 470)	$1 530/year[d]	
Southwest site	Storage and facultative lagoons	110	(416)	337 700	3 070	(0.81)	750	(1 850)	$5 780/year[d]	
Springfield, Illinois	Oxidation lagoon	67	(254)	176 000	2 600	(0.69)	80	(200)	$2 100/year	

[a] ENR = 2000

[b] Includes estimate of final clarifier.

[c] Includes plastic media trickling filter, final clarifier, plus one-half of other costs.

[d] Based on estimated man-day labor requirements.

Costs also include pumping, disinfection, and algae control systems where applicable. Engineering, administration, and land costs are not included in the estimates. Land costs may be the controlling factor in the evaluation of lagoon treatment systems and therefore must be assessed for each specific location.

Many biological treatment systems are integrated with or are a part of dry weather treatment facilities. Cost estimates of the wet weather portion of these facilities were separated from total costs of the complete treatment systems. The cost of the in-line RBC at Milwaukee, Wisconsin, was used together with an estimated cost for a final clarifier to develop the cost of a complete RBC treatment system.

Costs of lagoon treatment systems vary widely, and are a function of the type of lagoon, the number of cells, and the miscellaneous equipment requirements.

Costs of disinfection systems used to treat combined sewer overflows and storm water discharges can vary greatly depending on the complexity of the system. Cost curves comparing chlorine gas, chlorine dioxide, and hypochlorite generation disinfection systems are presented in Figure 62. These costs (ENR 2000) include manufactured equipment, piping, housing electrical and instrumentation, and miscellaneous items. No allowance for contingency or land was included. Operation and maintenance cost curves have also been developed.

To date, no summary cost data for solids handling and disposal are available. Costs have been estimated for individual operating facilities [46].

5.4.7.2 Variations in cost capacity utilization. Total operating costs consist of amortized capital cost plus operating and maintenance (O&M) costs. Amortized capital costs and a significant fraction of O&M costs are fixed on an annual basis. Since these costs must be assigned to the total volume treated, fixed costs per unit volume treated decline as capacity utilization rises. Capacity utilization (capacity used/capacity available) can be quite small for wet weather treatment processes in the absence of upstream storage. For example, in the preceding section, some of the treatment costs were based on 300 hours per year of operation, which corresponds to a capacity utilization of about 3.5%.

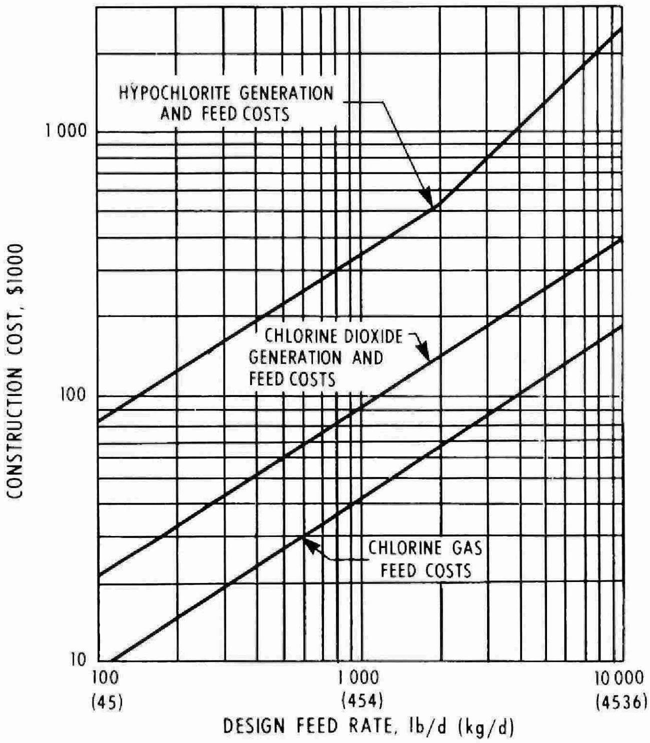


FIGURE 62. CHLORINE DISINFECTION COST CURVES (ENR 2000) [49]

Figure 63 illustrates how total operating costs decline as annual hours of operation increase for a high-rate screening process [64]. There is a four-fold reduction in total operating costs as annual hours of operation increase over the range 250 to 7 000 hours.

The integration of wet and dry weather treatment facilities and/or dual use can provide substantial reductions in the overall community investment in pollution control.

5.4.8 Case studies

This secton describes four pilot or full scale applications of CSO treatment techniques. These case studies illustrate combinations of the processes discussed. Storm water case studies are not included since full scale treatment to date has consisted only of sedimentation ponds.

5.5.8.1 Screening and high-rate filtration (Brooklyn, N.Y.). A program to investigate the effectiveness of fine screening followed by high-rate filtration to treat wet and dry weather flow from combined sewers was carried out at Newton Creek WPCP, Brooklyn, N.Y. This program followed successful smaller scale investigations.

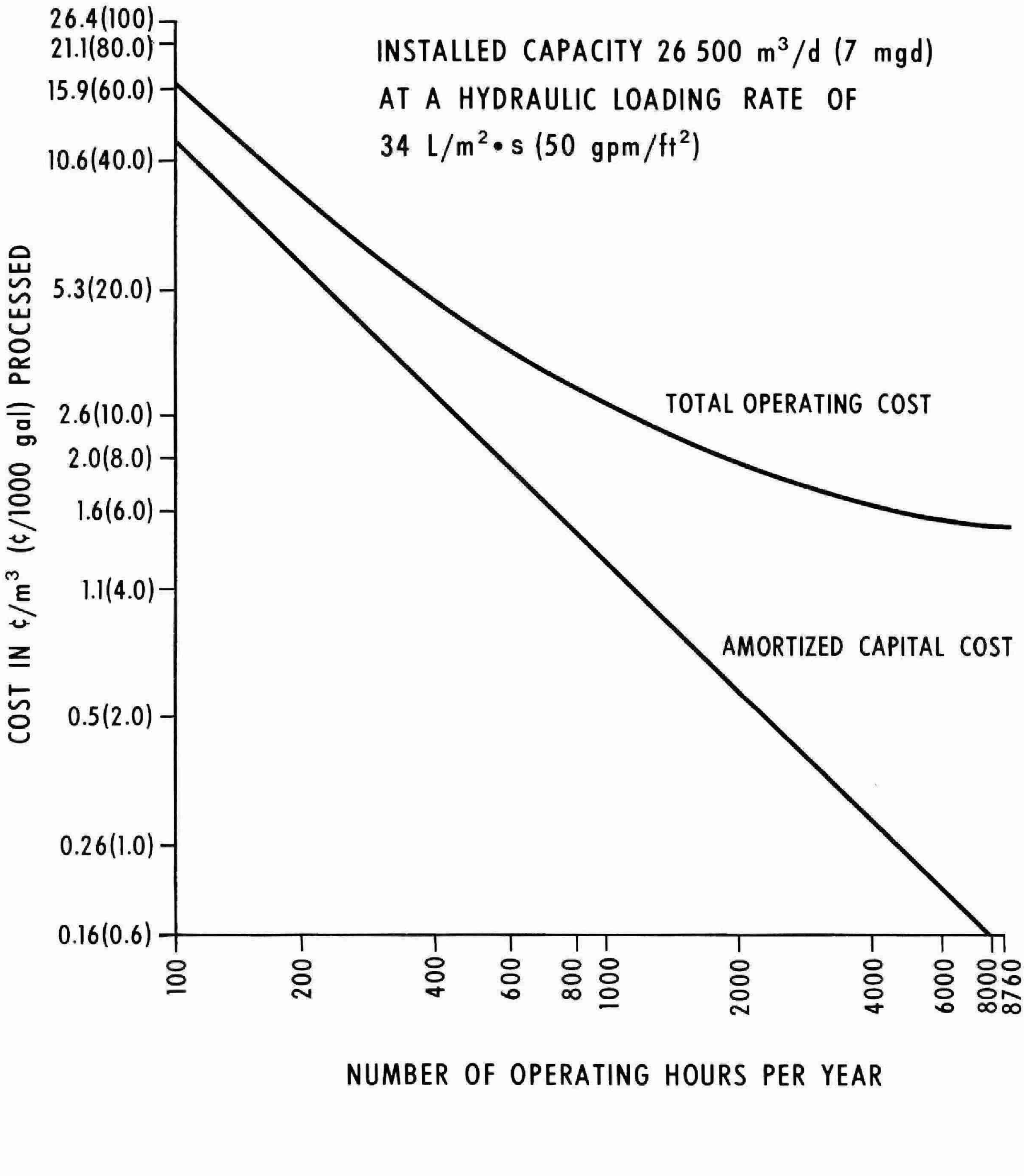


FIGURE 63. CWC SCREEN - AMORTIZED CAPITAL COSTS AND TOTAL OPERATING COST vs TOTAL ANNUAL HOURS OF OPERATION [64]

Raw sewage, pretreated only by bar screens, was pumped through a "Disco" strainer (40-mesh screen) rated at approximately 6.3 L/s (100 U.S. gpm) and then passed to a holding tank with up to 10 minutes detention time (Figure 64). From the holding tank, sewage was pumped to the gravity downflow filters. Chemical injection of alum, polymers or both was made into the discharge of the filter feed pump.

The Disco strainer, manufactured by Hydrocyclonics, is shown schematically in Figure 65. It has only recently been applied to storm water. The unit consisted of a series of stainless steel woven wire discs mounted on a centre shaft. The discs were 40% submerged in water and rotated continuously at speeds between 5 and 15 rpm. The liquid flowed radially through the discs while suspended solids were retained to form a precoat. When the solids in the system reached a selected concentration, the excess portion was carried upwards by the revolving discs and automatically discharged from the chamber as a solids cake. A backwash system was incorporated to remove any accumulation of solids from the discs, and to prevent mesh blinding.

The filter consisted of a 5.5 m high x 1 m (18 ft x 3 ft) diameter acrylic column with 2.1 m (7 ft) of anthrafilt over 1 m (3 ft) of AWWA fine sand and 15 cm (6 in) of pebbles all supported on a grid containing backwash nozzles.

Under storm conditions, typical operating results for suspended solids removal were 67% without chemicals and 88% with chemicals, at loading rates of 11 $L/m^2 \cdot s$ (16 U.S. gpm/ft^2). Normal run time at this loading was four to six hours to a terminal head loss of 2.1 m (7 ft). Typically, the filtration rate declined only in the last 30 minutes. The backwash rate was about 14 $L/m^2 \cdot s$ (20 gpm/ft^2) for six minutes with air scour before backwash. Chemical dosage for 88% suspended solids removal was 40 mg/L alum and 1 mg/L polymer.

5.4.8.2 Flotation and drum screening (Racine, Wisconsin). Combined sewer overflow was recognized as a significant pollution source by the City of Racine, Wisconsin. Most of the combined sewers in Racine are located on the north side of the city in a 280 ha (700 acre) area. In 1970 it was estimated that sewer separation for the entire area would cost $10 to $13 million, while satellite treatment plants using drum screening and

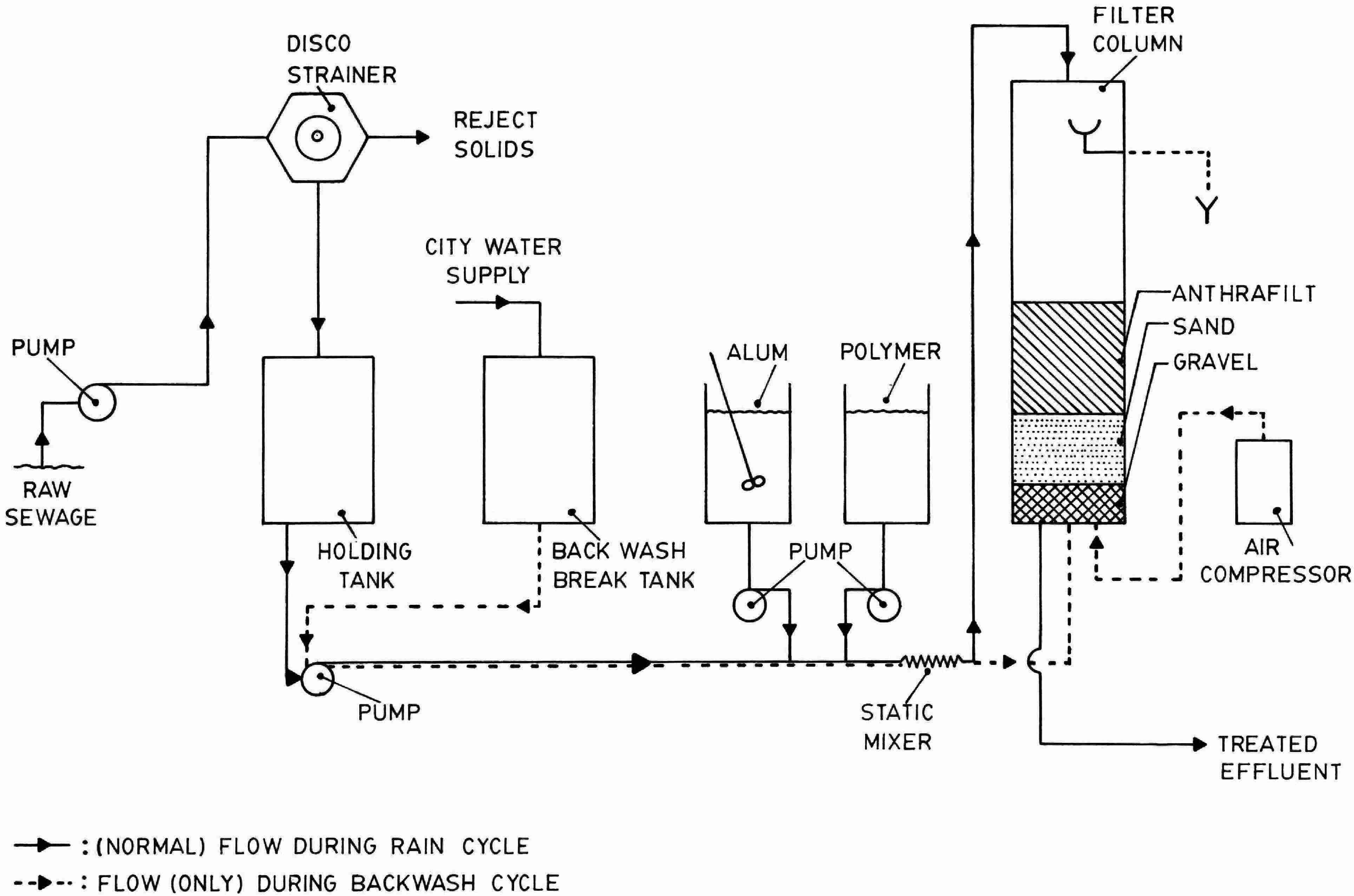


FIGURE 64. SCHEMATIC OF HIGH-RATE FILTRATION PROCESS

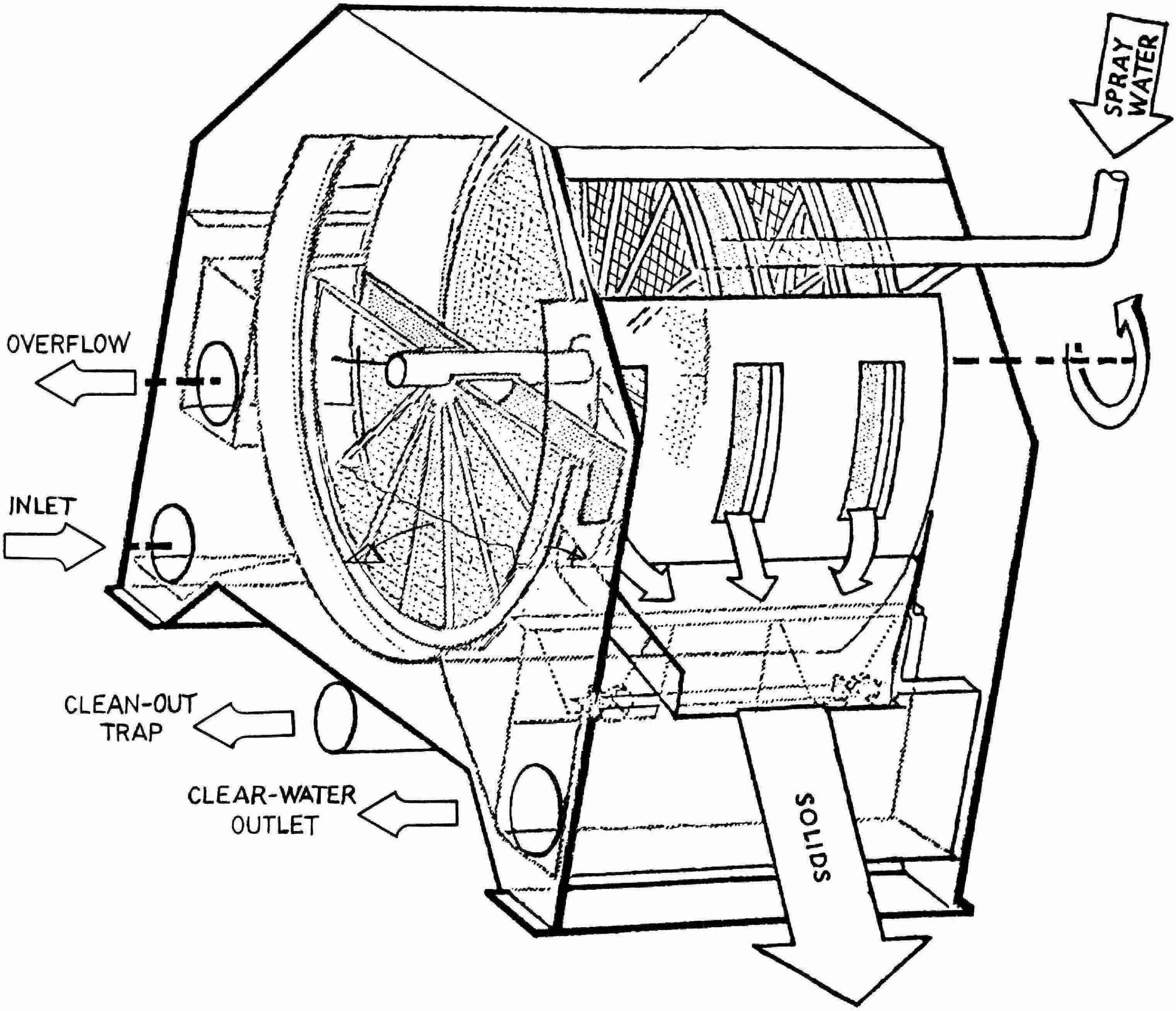


FIGURE 65. ISOMETRIC VIEW OF A DISCOSTRAINER

flotation would cost \$4 million. To demonstrate the feasibility of the latter concept two satellite plants of similar design, with capacities of 53 370 m^3/d (14.1 U.S. mgd) and 168 000 m^3/d (44.4 U.S. mgd) respectively, were constructed and operated.

A schematic diagram of the treatment process is shown in Figure 66. After entering the wet well, combined sewer overflow passes through a bar screen and is pumped into a chamber containing the drum screens. These screens have 50-mesh apertures and are designed to remove the larger suspended solids. The screens are backwashed with screened wastewater. After chlorination, the screened effluent flows into the flotation tanks where it is combined with a portion of the flow which has been pressurized. Ferric chloride and cationic polyelectrolytes are added to enhance flotation.

Sludge on the surface of the tanks is skimmed and conveyed to a sludge holding tank. This sludge is released back to the interceptor sewer after overflow has ceased. The plants are completely instrumented so that automatic start-up and shut-down is possible.

During system operation some grit deposition occurred in the flotation tanks, and in future designs, bottom scrapers may be advisable to remove sediment periodically.

5.4.8.3 Contact stabilization (Kenosha, Wisconsin). A 75 700 m^3/d (20 U.S. mgd) contact stabilization plant for treatment of CSO was constructed for the City of Kenosha, Wisconsin in 1971. The treatment system was added to an existing 87 000 m^3/d (23 U.S. mgd) conventional activated sludge plant operated by the Kenosha Water Utility.

Wet weather treatment facilities were constructed at the WPCP because the intercepter sewer conveying wastewater to the treatment plant had capacity well in excess of dry weather flow. WPCP design capacity was 87 000 m^3/d (23 U.S. mgd), while interception capacity was approximately 189 000 m^3/d (50 U.S. mgd), dry weather flows were about 75 700 m^3/d (20 U.S. mgd), with only a small diurnal variation, since a large proportion of this flow came from an industrial discharge of cooling water. With the addition of the contact stabilization plant, total treatment capacity during wet weather was increased to 163 000 m^3/d (43 U.S. mgd). The \$1.5 million demonstration system had the following components: raw

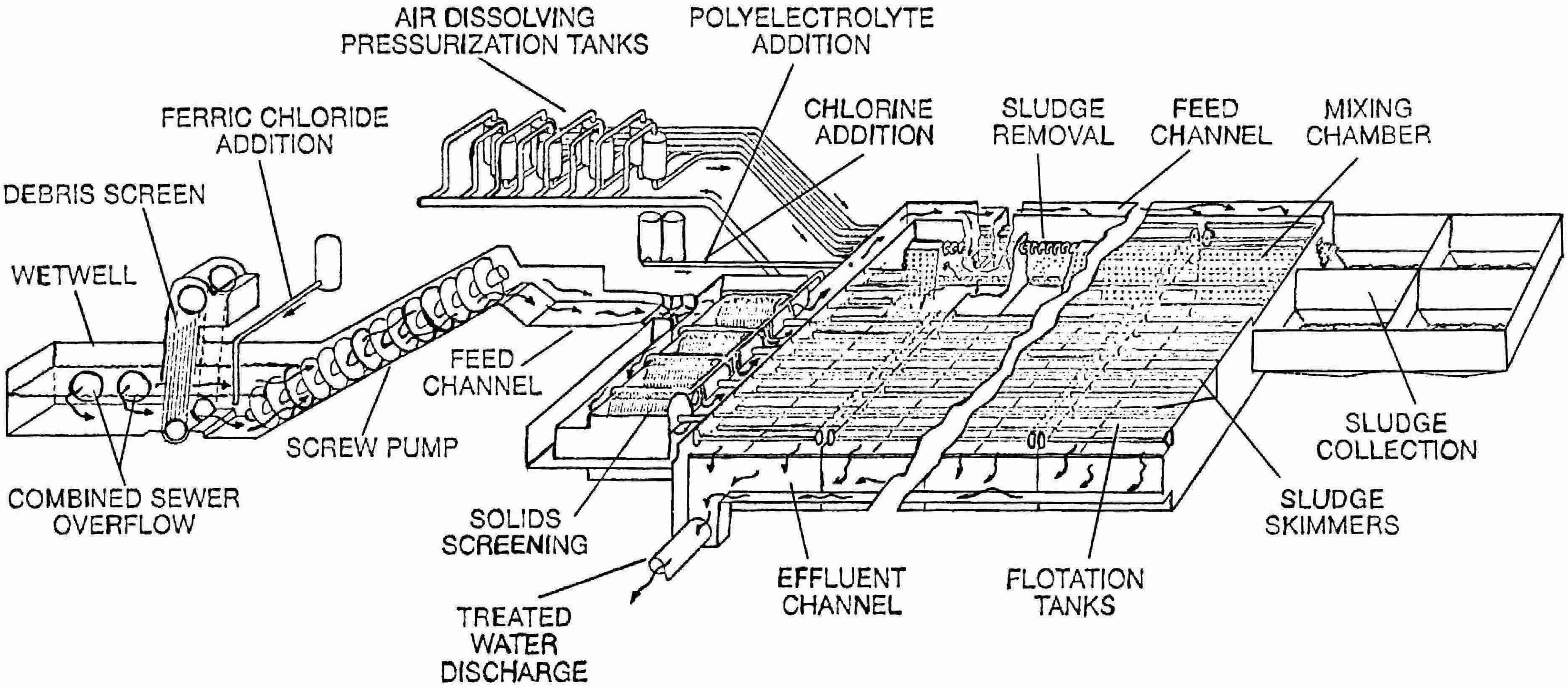


FIGURE 66. SCHEMATIC OF SCREENING/DISSOLVED-AIR FLOTATION PROCESS

sewage pumps, grit chamber, two stabilization tanks, two contact tanks, a final clarifier, and controls and instrumentation.

Innovations in plant design were the method of integrating wet and dry weather plant operations, and the methods adopted to bring the contact stabilization plant into and out of service rapidly and maintaining an active biomass during dry weather.

During the two-year demonstration period the system was operated in 46 potential overflow events, removing a total of 214 300 kg (472 500 lb) suspended solids and 92 400 kg (203 800 lb) of BOD [65]. This is equivalent to 20-25 days of raw sewage discharge from the city of Kenosha. Average reductions were 92% for suspended solids, 83% for BOD, and 80% for TOC.

Following the demonstration period, the plant was operated a further year by the Kenosha Water Utility. Since operation was not mandatory, it was discontinued in the fall of 1975, once the maximum useful experience had been gained.

A number of operating problems were experienced with the plant, such as icing of mechanical aerators in prolonged cold weather. Solutions could be readily incorporated in the design of a new facility as could the use of storage to optimize the total cost of treatment.

Presently, the City of Kenosha is evaluating an overall strategy for reducing CSO, which will include consideration of the information obtained from the demonstration project. The cost of separating combined sewers in Kenosha was estimated at $20 million. A contact stabilization system to service the whole of the Kenosha area was estimated to cost $8 million.

5.4.8.4 Multiple use lagoons (Mount Clemens, Michigan). A demonstration project utilizing a lagoon and recreational lake concept was undertaken for the City of Mount Clemens, Michigan, to solve a combined sewer overflow problem [66]. Following the successful demonstration of the concept, the city decided to develop a wastewater collection system and multipurpose treatment facility to reduce CSO overflows to the Clinton River.

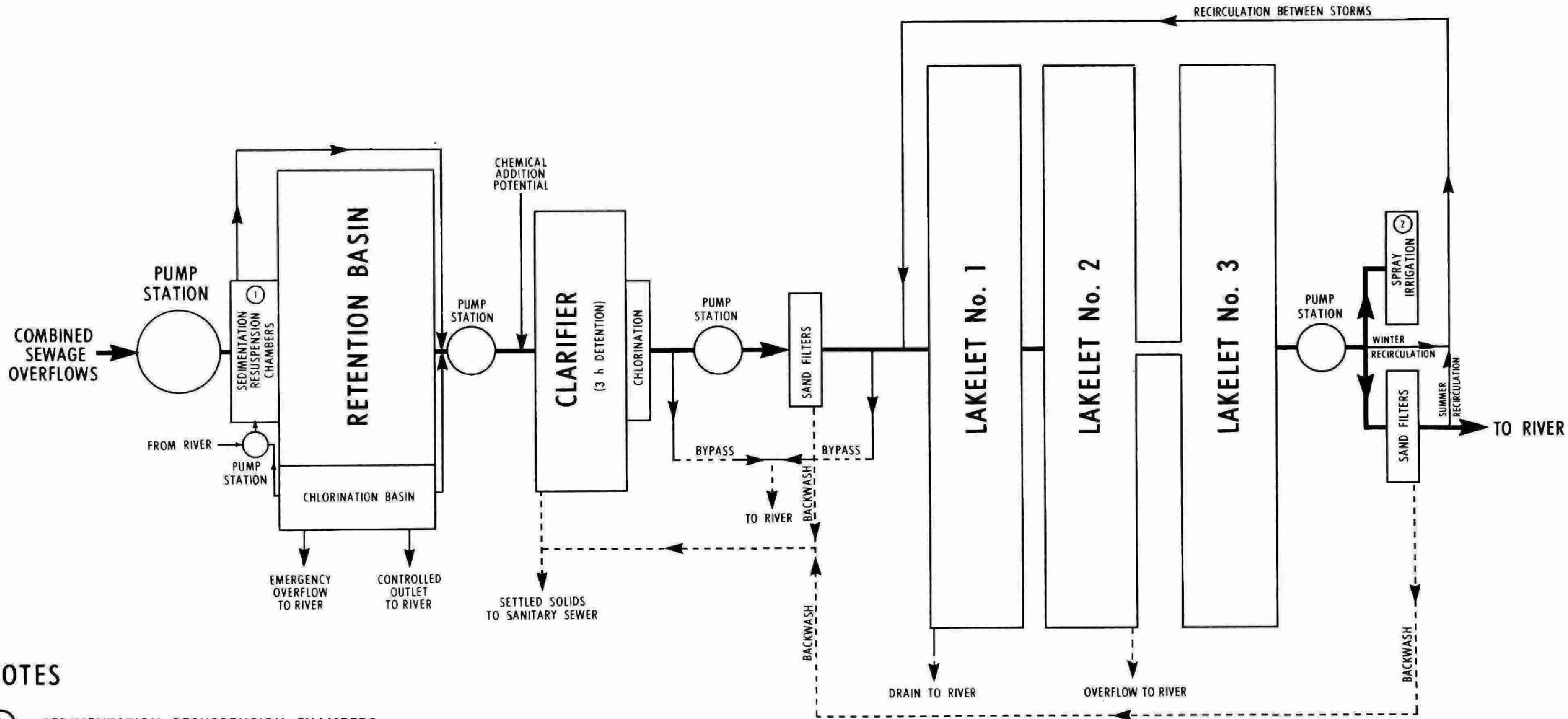
A combined sewage interceptor was built for a substantial portion of the city, 600 of a total 850 ha (1500 of 2100 acres), to collect overflows and convey them to a retention basin. Interceptor design was

based on a five-year storm with a runoff coefficient of 0.5 and time of concentration of $2\frac{1}{4}$ hours. For the remaining portions of the city a sewer separation plan was adopted. Dry weather sewage will flow to a regional wastewater treatment plant.

The collection and treatment scheme proposed for CSO is shown in Figure 67. After passing through sedimentation resuspension chambers (SRC), the overflows are introduced to a 1.2×10^5 m^3 (33×10^6 gal) retention basin designed to retain a five-year design storm. Aeration is provided in the retention basin by floating mechanical aerators. When the retention basin capacity is exceeded, the excess will overflow into the chlorination basin and be discharged to the Clinton River. The retention facility provides a minimum of three hours residence time during sedimentation and aeration. The retention basin is followed by a clarifier, chlorination chamber, and sand filters. Flow from the clarifier is set at a constant rate of 15 140 m^3/d (4 U.S. mgd). Effluent from the sand filters enters a series of three lakelets which provide 4.7 days of detention time at the influent flow of 15 140 m^3/d (4 U.S. mgd). Submerged aeration is provided in all the lakelets.

The projected effluent quality of the facility is based on the results of the demonstration project and is expected to meet all the requirements. The demonstration project showed that the complete treatment system reduced suspended solids from 350 mg/L to 14 mg/L and BOD from 140 mg/L to 6 mg/L. A phosphorus removal system may be installed ahead of the clarifier for algae control. It is expected that 70% of the suspended solids will be removed in the clarifier and transported as sludge to the regional wastewater treatment plant. The remaining solids are expected to settle out in the lake system where aerobic and anaerobic digestion will eliminate the problem of sludge accumulation. The third lake in the system is expected to be suitable for secondary contact recreation. The lake system will always be filled, providing a flow-through treatment process rather than a fill and drain operation.

The estimated capital cost for the total project is \$16.87 million. The annual ammortization costs of \$1.59 million and the annual treatment cost of \$835 000 result in a total annual cost of \$2.43 million.



NOTES

① SEDIMENTATION RESUSPENSION CHAMBERS AT 50% OF PEAK RATE HAVE 1 h DETENTION

② SPRAY IRRIGATION: 5.8 ha

FIGURE 67. FLOW DIAGRAM - RETENTION, TREATMENT, AND LAKELETS AT MOUNT CLEMENS, MICHIGAN [66]

5.5 Evaluating Alternative Solutions

5.5.1 Approach

Preceding sections of this chapter have discussed collection, storage, and treatment as individual control measures. In practice, these methods should be used in combination to achieve the desired objectives at minimum cost.

Both remedial programs in existing areas and proposed programs for new development are handled best within the framework of a master drainage plan and the municipality's official plan.

The purpose of a master drainage plan is to ensure that storm water drainage systems are developed in a manner compatible with watershed needs, that existing problems are properly identified, and that future drainage problems are avoided. The effects of drainage proposals associated with new developments can be assessed most efficiently and speedily if master drainage plans have already been prepared for the entire municipality and especially for the receiving watercourse.

While master drainage plans are developed by municipalities and thus follow municipal boundaries, inter-municipal coordination is necessary to resolve drainage problems, particularly where watershed boundaries cross municipal boundaries.

The end result of the master drainage plan is a set of specific objectives indicating the uses that must be maintained along designated sections of watercourses with a general outline of preventative or remedial measures required to meet these objectives. A more detailed pollution control strategy would define a combination of remedial measures to achieve the appropriate level of control over pollutant discharges from all sources to meet receiving water objectives. Once formulated, the strategy provides the basis for future water pollution control expenditures by the municipality, subject to periodic updating.

Preparation of the strategy begins with an overall analysis to determine the relative magnitude and distribution of major water pollution problems in the municipality for present and future conditions, and an in-system review to indicate sewer system characteristics and wet and dry weather sources of pollution. This preliminary analysis relies on existing or readily available data, and will indicate those areas requiring detailed investigation and data collection programs.

The maximum allowable discharge loadings of critical pollutants should be established in conjunction with the Ministry of the Environment through master drainage planning and/or waste assimilation studies based on the Provincial Water Quality Objectives.

To ensure a comprehensive approach, an evaluation of loading effects and the achievable level of control of all significant pollution sources should be made, including as applicable:

- sewage treatment plant effluent discharges,
- sanitary sewer by-passes (in the sewer system and at the sewage treatment plant),
- storm sewer discharges (including fugitive and illegal discharges),
- combined sewer overflows,
- industrial waste discharges, and
- other non-point sources within the municipality.

The efficiency and effectiveness of current and proposed sewage treatment processes is then evaluated for both dry and wet weather conditions. When evaluating wet weather performance, the effects of anticipated variations in influent quantity and quality, due to inflow-infiltration or storm flows, should be considered. Systematic analysis of wet weather performance is mandatory where advanced treatment is required or when the Ministry of the Environment has determined that variability of waste strength and high hydraulic peak flows are adversely affecting the existing treatment plant performance.

Wet weather control measures can include sewer construction, deep tunnels, off-line storage, treatment, or attempts to maximize the performance of the existing system through non-structural controls such as sewer flushing, cleaning and maintenance. Since non-structural controls can be implemented more rapidly than structural solutions such as new conveyance and downstream treatment facilities, they may be used as interim measures. Although non-structural controls are much less capital intensive and less disruptive than large-scale construction, they typically have higher operating and maintenance costs. Thus cost trade-offs can be made. In developing strategy, a balanced approach is desirable using non-structural controls and investigating dual-use

facilities where possible. When wet and dry weather pollution control are considered together, trade-offs can be made between advanced treatment of dry weather flows and control of storm flows. In evaluating alternatives, consideration should be given to flood control benefits. In some cases, multiple benefits and cost savings can be obtained by staging construction, e.g., coordinating collection system changes with road improvements.

For new developments, the emphasis should be on prevention of storm water pollution problems, as discussed in previous sections of this manual. This will require good planning and good analytical tools with an emphasis on natural drainage and source controls.

5.5.2 Case studies

Case studies illustrating the evaluation of alternative solutions are presented in this section.

5.5.2.1 City of St. Thomas, Ontario. A recent study for the City of St. Thomas [67] included a comprehensive analysis of pollution control and sewer system flood relief alternatives. The study objective was to identify cost-effective solutions to interrelated wet weather runoff problems such as basement flooding, pollution from combined and storm sewer discharges and WPCP by-passes. These problems existed with a severely limited assimilative capacity in the receiving stream, Kettle Creek, the consequent possible need for advanced treatment of dry weather flows, and a desire to arrive at solutions which would permit continued urban growth.

St. Thomas has a population of 27 000 and an area of 1820 ha (4500 acres). Land use is 44% residential, 14% industrial and 7% commercial. The balance is open space, mainly ravine land, Kettle Creek Valley, and parkland. About 25% of the total sewered area of 117 ha (2900 acres) is served by combined sewers, the balance being served by separated sanitary and storm sewers.

At the beginning of the study, St. Thomas was about halfway through a comprehensive sewer separation program. As a consequence, the sewer system includes sanitary, storm, and combined sewers with many interconnections. In general, both the existing sanitary and combined sewers are inadequate for conveyance of storm flows. Sanitary sewers

receive drainage from footing tiles which are connected throughout the city, while combined sewers receive roof leader flows in some areas.

Analysis of the sewer systems showed basic hydraulic inadequacies in the combined sewers and a lack of significant in-sewer storage. Completion of the sewer separation program, and disconnection of roof leaders to the maximum extent possible were recommended as the most practical measures for reducing flooding in the existing system. Inlet controls and on-site storage offered the prospect of achieving cost savings in sewer construction or reconstruction for new development or redevelopment.

Using a combination of monitoring and modelling to arrive at pollution loadings, it was determined that during the critical period of April to October, both wet and dry weather discharges had a significant impact on Kettle Creek. Forty-six percent of BOD discharges were from the WPCP, 30% from combined sewer overflow and 24% from separate sewer discharges. Two-thirds of all suspended solids discharges were from separated storm sewers.

The analysis of alternatives to reduce wet weather loadings identified improved street sweeping practices and elimination of combined sewer overflows by sewer separation as the most cost-effective solutions. These combined measures would reduce wet weather BOD and suspended solids discharges by approximately 90%.

Diversion of secondary effluent from Kettle Creek to Lake Erie was recommended for reduction of dry weather pollutant discharges. This would necessitate construction of an effluent pipeline as part of a regional wastewater collection and treatment scheme.

Overall costs were developed and a staged implementation of various measures was recommended.

5.5.2.2 Town of Midland, Ontario. The Municipal Council of the Town of Midland, Ontario commissioned a study [68] to assess the development potential of the Midland Park Lake (Little Lake) watershed.

The objectives of the study were to determine the impact of additional storm water runoff from further development, and to develop and cost the storm water management measures needed to maintain water quality suitable for existing recreational uses including fishing and swimming.

With only 15% (97 ha-240 acres) of the watershed urbanized, water quality in Little Lake was generally very good, and it had been concluded that significant problems were unlikely to develop in the absence of further urbanization. Inputs from storm sewers were known to be a contributing factor to fluctuating coliform counts at local bathing beaches, which had become only marginally suitable for swimming. No treatment plant effluent is discharged to the lake.

The storm water management study included an evaluation of Little Lake hydrogeology and the development of a water budget for the lake and watershed. The lake is perched above the water table and more than half the water lost in a normal year is by leakage through the bottom, compared with 9% lost by seasonal outflows. Leakage is augmented by wells which provide the municipal water supply. Storm water runoff was needed to maintain a satisfactory lake water budget.

Storm water was identified as the major controllable source of bacteria and phosphorus, and a monitoring program was conducted to determine the quality of storm water runoff and the routes by which bacteria and nutrients could reach the lake. A phosphorus budget was developed since phosphorus was identified as a critical pollutant.

The study found that without storm water treatment, a further 50 ha (125 acres) of low-density residential land could be developed. With storm water treatment by storage-sedimentation followed by alum coagulation for phosphorus removal and bacterial reduction, a total of 200 ha (500 acres) could be developed. Controls would cost $500/ha/yr ($200/acre/yr). The developed area permissible and control costs for various mixes of higher-density development were also calculated.

Means of minimizing phosphorus input to the lake at relatively low cost were identified. They included measures to keep catch basins and streets free of decomposing vegetation, and a policy of protecting a swamp at the west end of the lake which appeared to be acting as a sediment trap.

5.5.2.3 City of Nepean and Township of Gloucester. The City of Nepean and the Township Gloucester plan to develop a south urban community on lands adjoining the Rideau River south of the City of Ottawa. This development is projected to have a total population of 100 000 on an area

of 2200 ha (5400 acres). Runoff from the area would be discharged to the Rideau River. With only limited assimilative capacity in the river, the Ontario Ministry of the Environment has stated that further development along the Rideau River must "be based on the principle of no deterioration to the existing quality of the watercourse" [69]. For large new developments, a report is required on the effect of storm drainage on the area's watercourses and the proposed methods of control. To fulfill this requirement, a study was performed which evolved a conceptual plan for storm water management for the south urban community, described the staging of construction to implement the concept, and estimated costs.

The Ministry of the Environment guidelines for storm water discharges to the Rideau River and receiving water objectives are presented in Table 47.

TABLE 47. ONTARIO MINISTRY OF THE ENVIRONMENT RIDEAU RIVER STORM WATER GUIDELINES [69]

Parameter	Effluent Objective	Receiving Water Criteria
BOD	5 to 6 mg/L	4 mg/L
Total Phosphorus	1 mg/L	--
Chlorine	0.1 mg/L	0.02 mg/L
Total Coliform	1000/100 mL	1000/100 mL
Fecal Coliform	100/100 mL	100/100 mL
Fecal Streptococci	20/100 mL	20/100 mL
Suspended Solids	maximum removal possible	--

Analysis showed that, with no controls on runoff, the predicted total average annual loadings from the proposed development would exceed the guidelines established by the Ministry of the Environment.

A number of storm water management control techniques were investigated, including such source controls as improved street sweeping

practices, lot grading and drainage practices to retard and attenuate peak flow rates, and use of impervious areas for detention of flow and associated reductions in the rate and extent of pollutant wash-off.

It was recommended that street sweeping practices be implemented to provide a maximum interval between sweepings of two to three days for commercial areas and two weeks for other land use categories. It was also recommended that maximum lot gradients be reduced to 1% where practicable and that industrial sites be required to store runoff on large impervious areas such as rooftops and parking lots. A maintenance program would be required to remove solids and debris from the storage areas on a regular basis.

Although such source controls would have a beneficial effect on both quantity and quality of runoff, it was not anticipated that they would be sufficient to provide a discharge quality which would meet MOE guidelines. It was concluded that a system of storm water retention ponds would also be required. The proposed ponds would provide an average of 24 hours retention.

On the basis of model simulations using STORM and SWMM, it was estimated that implementation of the recommended site controls and construction of storm water retention ponds would produce storm water discharges substantially in accordance with the quality guidelines established by the MOE.

The additional cost to provide a system of storm water retention ponds integrated into the trunk sewer design was estimated to be approximately \$2000/ha (\$800/acre).

It was recommended that initial development in any of the sub-drainage basins include construction of a downstream retention pond for that basin to control erosion during construction.

5.6 References

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6 ADMINISTRATION AND IMPLEMENTATION OF URBAN DRAINAGE SOLUTIONS

6.1 Introduction

The previous chapters have discussed the problems associated with urban drainage, a general planning approach for solving urban drainage problems, methods for analysing urban drainage quantity and quality, and techniques for controlling and managing urban drainage. To ensure the effective implementation of these concepts and related technical considerations, the administrative and legal framework must permit and encourage their use. Only then can the stated goals and objectives of storm water management be achieved. In this context the administrative framework encompasses the full range of the planning and development, and pollution control processes.

A basic theme of this chapter is that urban drainage and the associated environmental concerns should be considered at all levels of the planning and development and pollution control processes from earliest stages to completion. At each level the input may vary but the aim is common. Ideally, the administrative framework should encourage the integration of urban design with urban drainage design to achieve the best overall solutions. Who regulates and who is responsible for various aspects of urban drainage planning, design, and control must be clearly recognized.

This chapter briefly reviews common and statute law affecting urban drainage in Ontario. The present development control process in the Province of Ontario and the method for considering urban drainage concerns are discussed. It is suggested that the existing framework is sufficiently comprehensive but that greater recognition of urban drainage is required throughout the planning and development and pollution control processes by all involved. A discussion of funding programs applicable to urban drainage and the availability of such funds is also presented.

6.2 Drainage Law

6.2.1 Common law

Most of the principles of common law are very old, and were developed by the courts before statutory provisions regarding drainage

were enacted. However, the principles of common law continue to apply, unless they are specifically altered or overridden by statutory legislation.

The common law regarding drainage may be divided into two parts: the rules governing the riparian rights and obligations of landowners whose lands are immediately adjacent to natural watercourses, and the rules governing the rights and obligations of landowners relating to surface or percolating waters.

Any landowner whose lands abut a natural watercourse has a right to drain his lands into that watercourse. A watercourse is a defined channel but does not necessarily have to contain a continuous flow. A landowner with lands abutting a natural watercourse who collects the rain in ditches or drains has the right to discharge it to the watercourse. This is so even if the result is to increase the volume or rate of flow of the stream. The downstream landowner must accept the possibility of increased flow of the stream since the upstream owner has the advantage of reasonable use of the stream for drainage. Reasonable use suggests use up to the capacity of the banks of the stream.

It is the duty of anyone who interferes with the course of a natural stream to see that the works which he substitutes for a natural channel are adequate to carry the water which may result from even an extraordinary rainfall. If damages result from improperly substituted works, provided in place of the natural stream, then the owner of the works is liable.

The riparian landowner is limited in that he may not bring waters into a natural watercourse which have not fallen upon the lands located in that watershed. Thus, water from one watershed may not be diverted into another. Similarly, the riparian landowner may not sell or assign the right to drain into a natural watercourse. That is, the owner of lands remote from the watercourse cannot buy or secure from a riparian landowner by common law the right to drain water into that natural watercourse.

The second division of common law deals with the water that descends in the form of rain, and finds its way by percolation, or flow to a natural watercourse. A landowner cannot claim the assistance of the

law to prevent the natural flow of surface water from adjoining high land, but also is not obliged to receive surface water flowing upon such adjacent lands. The low landowner may, without liability, protect his land by building structures, or by filling the land for protection, and the upper landowner has no complaint if flooding results on the upstream property.

In law, when water is collected in a man-made channel its character changes. A person who collects water in an artificial channel loses any right he might have had in respect to uncollected surface water. When he does so, he is responsible to avoid discharging his collected water on the lands of another, and he must at his expense, take the water to a sufficient outlet.

In both cases above, the basis is that of the rights and obligations of landowners. It should be noted that all landowners are considered equal whether they be a municipal corporation or a private citizen.

The common law forms the basis for drainage law in Ontario and always applies except when and where it has been varied by Federal or Provincial statute law.

6.2.2 Statute law

Since statute law takes away common law rights, such statutes must be interpreted strictly. Where there is a conflict between two acts, the legislation that is of a specific nature prevails.

6.2.2.1 Federal legislation. Under the division of powers made by Section 92 of the British North American Act, the fields of property and civil rights became the legislative responsibility of the provinces. This authority includes the law relating to land drainage.

There are, however, some exceptions. Section 92 provides that works and undertakings which extend beyond the boundaries of one province and work declared for the "general advantage of Canada" fall under exclusive federal jurisdiction. Such matters are not subject to provincial legislation except where federal legislation specifically makes such provincial legislation applicable. Also, "Indians and land preserved for the Indians" are under exclusive federal control. International Acts such as the Boundary Waters Treaty Act also fall within federal authority.

The most relevant federal legislation is the Fisheries Act, which contains general provisions prohibiting the deposit of deleterious substances in waters frequented by fish. This applies to deleterious substances from any source that could reach such waters by any manner. Thus, a deleterious substance that was deposited or permitted to be deposited by storm water or runoff would be contrary to the general pollution provisions of the Fisheries Act.

The Canada Water Act, which provides for joint federal-provincial watershed management planning and implementation of resulting study agreements, has the potential for use in solving urban drainage problems.

6.2.2.2 Provincial legislation. In Ontario, various forms of drainage statute law date back to 1835 and many pieces of existing legislation contain provisions relating to drainage. Due to the variety of legislation and its evolution a complex situation exists today.

The existing legislation in Ontario provides the opportunity for the consideration, regulation, and control of drainage in a comprehensive manner. The statutes to be considered include those concerned with planning and development, municipal authority, water management, and environmental quality. Obviously, a number of areas overlap in these categories of legislation and the situation may appear complex. However, to achieve the optimal solutions for urban drainage and design, problems, the concepts and environmental constraints of storm water management must be recognized throughout the planning, development, and pollution control processes.

The following is a summary of pertinent legislation affecting drainage in the Province of Ontario.

1) The Environmental Protection Act is an act to provide for the protection and conservation of the natural environment. The act has wide-ranging powers to control any possible activity of man which might directly or indirectly injure the environment since everything is included which might cause pollution in the definition of "contaminant". Under these provisions, urban runoff is defined as a "contaminant".

Under this legislation the Ministry of the Environment can prohibit or regulate the discharge of any contaminant as long as it can be proven that the contaminant can impair water uses and cause damage to life and property.

2) The Ontario Water Resources Act is intended to protect the water resources of the Province of Ontario. In this act, the term "sewage" includes drainage, storm water, and commercial and industrial wastes, and the term "sewage works" means any works for the collection, transmission, treatment, and disposal of sewage. The approval of "sewage works" is required under the act and a person or municipality can be ordered to establish, maintain, operate, improve, enlarge, repair, etc., any sewage works.

3) The Environmental Assessment Act provides for the protection, conservation, and wise management of the environment in Ontario. The act requires that an environmental assessment of proposed projects and activities be prepared by the proponent to consider in part the potential effects of the activities on the environment and the means by which the effects could reasonably be mitigated. In the first phase the act applies to provincial projects. The act has been proclaimed for the private sector but will be applied by regulation on a case by case basis. The conservation authority projects came under the act by a regulation dated September 1, 1977, and are subject to phased implementation. The municipal sector will come under the act in the future, and such municipal projects will be phased in.

4) The Conservation Authorities Act provides for the establishment of conservation authorities for the purpose of establishing an undertaking, in the area over which they are given jurisdiction, a program designed to further the conservation, restoration development, and management of natural resources other than gas, oil, coal, and minerals. The focus of their activities is water management for flood control.

5) The Planning Act provides for the adoption of an "official plan" by a municipality which will identify programs and policies of that area designed to secure the health, safety, convenience, and welfare of the inhabitants of the area. In consideration of physical, social, and economic conditions the official plan has a very wide scope that would include storm water management policies and programs. No public works shall be undertaken which do not conform to the official plan. The act also provides for the development of land by plans of subdivision and consideration is to be given to conservation of natural resources and flood control in reviewing these plans. The municipality may enter into subdivision agreements which can specify restrictions which will affect the drainage or the urban area.

   The Planning Act gives municipalities the right to pass bylaws to control the use of land and the manner in which land is developed.

6) The Municipal Act contains a number of provisions respecting drainage. A municipality normally has power to act only within its boundary, unless there is some statutory authority otherwise. The Municipal Act enables councils of municipalities to enact bylaws which may relate to drainage and water control.

7) The Local Improvement Act permits a municipality to undertake a large variety of "works", and to assess those homeowners who directly benefit from a much larger share of the cost than would be the case if the works were financed out of the general funds of the municipality. Major drainage works and shore protection can be included. New, extended and improved works are covered by the act, but the simple maintenance of existing works is not.

   The works may be undertaken by council initiative or by a signed petition of the majority (over 50%) of the affected homeowners. Works carried out under this act place a burden on a municipality's borrowing capacity, as the municipality must finance the works.

8) The Drainage Act is to provide for the construction, improvement, or maintenance of drainage works by municipalities. Grants may

be made in respect of assessment made under the act upon land used for agricultural purposes.

9) The Lakes and Rivers Improvement Act ensures the suitability of the location and nature of improvements in lakes, rivers, streams and watercourses, including their efficient and safe maintenance and operation.

10) The Beds of Navigable Waters Act provides for Crown ownership of the beds of navigable waters unless an express grant has been made to the contrary.

The recognition and consideration of storm water management through this range of activities and statutes require a high degree of understanding and cooperation on the part of developers, municipal officials, planning and engineering consultants, and provincial representatives. The greatest success in the protection of water resources and the selection of optimal storm water management solutions can be best achieved with an integrated watershed approach to urban and storm water planning. This planning approach was discussed in Chapter 2.

From an environmental protection and water quality point of view, the Province has sufficient authority in The Environmental Protection Act and The Ontario Water Resources Act to require comprehensive studies and facilities to control urban drainage. The Ontario Water Resources Act specifically relates to the control of storm water from the standpoint of water pollution. The Environmental Protection Act is less specific but provides a broad scope for the general protection and enhancement of the environment. However, approvals issued to date under The Ontario Water Resources Act have related mostly to hydrologic design aspects of a proposed storm sewer system. Only in very special cases has a wider range of concerns been applied.

Some municipalities may find it difficult to adopt an integrated approach to some aspects of storm water management. Urban drainage responsibilities have been assigned to both municipal and regional levels where regional government has been instituted. Similarly, where the Province maintains certain roads in a municipality, the Province is responsible for certain urban drainage aspects. For instance, maintenance and

operation functions, such as street sweeping, could be the responsibility of three levels of government in one municipality. This split jurisdictional problem must be overcome and can likely be best resolved through cooperation in the achievement of common objectives and programs.

Cooperation and a common acceptance of the importance watershed-scale planning are necessary when municipal boundaries cross a watershed. Higher levels of government such as a regional government, conservation authority and/or the Province can assist with appropriate policies, programs, and guidelines.

A higher level of government should assume the responsibility of providing coordination to achieve the optimal master drainage plan for the watershed.

6.3 Development Control

Urban drainage problems or effects are the result of changes in the type or intensity of land use. The environmental quality effects of urban drainage can be reasonably regulated by available pollution control legislation. However, a more comprehensive and more effective urban drainage solution can be achieved by incorporating urban drainage concerns into urban design and into the planning of land use change.

Pollution control legislation focuses on the effects of the urban drainage on the environment. This approach can be narrow and can result in expensive solutions. Mechanisms that control planning and development can consider land use and urban design more broadly and thus affect the input to the urban drainage system. The review of legislation affecting drainage suggests that a comprehensive planning and development control process is available which could accommodate a range of policy, regulation, and implementation approaches to storm water management.

The Planning Act, presently under review by the Province of Ontario, provides a statutory framework for land use planning and implementation at the local level. The act is administered by the Ministry of Housing and the Ontario Municipal Board, and the key responsibilities are the review and approval of official plans and amendments, subdivision plans, restricted area or zoning bylaws, and provision of advice to municipalities on planning matters. Many provincial agencies have a significant involvement in this process since

their advice is solicited on planning and development proposals and documents. Municipalities and concerned provincial agencies should ensure that adequate consideration of the control of urban drainage is provided in the land use control process at the time of review.

6.3.1 Official plans

The official plan for a municipality sets out the policies and programs which are to guide the development of the area. No public work can be undertaken and no bylaw passed which does not conform to the official plan. General and specific criteria can be included against which the environmental impact of a proposed work or activity can be measured. If the policies proposed by a municipality are not considered adequate by provincial agencies, the Minister of Housing may require or make alterations. Policies concerning storm water management should be included.

The official plan should state environmental, social, and economic goals or principles upon which storm water management systems in the municipality are to be based. At this general level of planning it is important for the municipality to recognize and to make a commitment to comprehensive urban drainage planning as a basic element of all stages of land use planning. More specifically, such concepts as natural drainage, and regional and watershed drainage planning and their general goals and objectives should be recognized and adopted. A requirement or a commitment in specific watersheds within the municipality (developing, redevelopment, remedial areas) could be made for a "Master Drainage Plan" for a developing area, a "Total Environmental Management Plan" for an urban area, and a "Flood Relief Study" within an existing urban area. These studies were discussed in detail in Chapter 2. Ideally, such studies could be an input to the development of the land use plan contained in the official plan.

6.3.2 Secondary plans

These are plans that provide detailed land use plans and policies for a portion of the area covered by the official plan. In development of these documents, watershed drainage studies should be performed as part of the evaluation of alternative land use proposals.

Specific policies and requirements should be stated for matters such as erosion control, site design, flood plain management, and protection of "hazard lands" or sensitive areas.

6.3.3 Restricted area and site plan control bylaws

Under The Planning Act, a municipal council can pass bylaws which restrict the use of lands. More specifically, in Section 35(a) the Act states that where an official plan is in effect the council, as a condition of development or redevelopment of land or buildings, can prohibit or require the provision, maintenance, and use of certain facilities and may regulate the maintenance of such facilities. Aspects of urban drainage control include: grading of land and the disposal of storm, surface, and wastewater from the land and from any buildings or structures, conveyance to municipality of easements for construction, maintenance or improvement of watercourses, ditches and land drainage works on the land, walkways, and landscaping. In general, such bylaws should adopt the storm water management concepts discussed earlier.

6.3.4 Subdivision plans

When a person wishes to subdivide his land and develop all or part of it, the land must be described in accordance with a plan of subdivision which has been registered prior to any development. The plan cannot be registered until a draft plan of subdivision has been approved by the minister or a municipality to which this authority has been delegated and all conditions placed on it cleared. The draft plan must indicate, in part, the nature of existing uses of adjoining land, natural features, soil conditions, and municipal services available or proposed. In considering approval, the minister, on the advice of appropriate provincial agencies, must have regard for many criteria including adherence to the official plan, conservation of natural resources and flood control, and the adequacy of municipal services. The minister can impose whatever conditions to the draft approval of the plan of subdivision he considers necessary. As an integral part of urban design, detailed storm management measures, such as storm water management criteria, urban design, and erosion control, should be considered at this stage.

In addition, the minister and any municipality may enter into agreements with a subdivider that impose conditions to the approval of a plan of subdivision which are enforceable against the owner and subsequent owners of the land. The agreement can include restrictions on short-term activities such as construction; specific design criteria for physical works; and identification of responsibilities for financing, design, construction, monitoring, maintenance, and ownership of works. These are among the most influential controls on development.

6.3.5 Municipal bylaws

The Municipal Act and other legislation assigns the power to municipalities to pass bylaws pertaining to a wide variety of matters. Bylaws respecting sewer use and plumbing, drainage, and building design are of particular concern to storm water management. Different standards and inspection requirements may apply in sewer use and plumbing bylaws and one set of requirements could conflict with the aims of the other in such areas as construction standards, inspection requirements, infiltration, discharge of foundation drains, materials, and the connection of roof downspouts. The appropriate municipal bylaws should be amended to conform with the overriding philosophy of urban storm water management.

To be effective a bylaw must be enforced by experienced municipal officials and their intent must be understood by the public. If enforcement is inadequate it does not matter how many bylaws exist or what they attempt to achieve.

6.3.6 Municipal engineering standards

Over many years, municipal engineering departments compile extensive engineering standards documents which specify design criteria for works within the municipality. These can be conservative and restrictive because they often develop from many piecemeal additions over the years in response to problems and difficulties that arise in specific developments. Few recognize in a comprehensive way the concepts of storm water management put forth in this report, and the standards should be reviewed and revised where necessary.

6.3.7 Summary

Through statutes such as The Planning Act, a comprehensive land development control process exists in Ontario. The regulation of land use can indirectly control the environmental impact of urban development and the management of urban drainage. Since drainage is directly affected by development and is an integral part of urban design, storm water management should be appropriately considered at each level of the planning and development process. It is only with such a comprehensive approach to urban drainage design that the most environmentally, financially, and socially acceptable solutions can be achieved.

Pollution control legislation is also applied to new development and redevelopment, and the pollution control and planning legislation should be complementary. For example, the Ontario Water Resources Act requires that approval be obtained for the establishment or extension of a sewage works. "Sewage" is defined to include drainage and storm water. Plans, specifications, and reports must be included in an application for approval. This report recommends that the submission of an evaluation and analysis which takes account of comprehensive storm water management concepts and the impact of urban drainage on the environment be considered as an additional requirement of the review and approval process.

Table 48 identifies storm water management aspects to be considered at various stages in the planning and development control process by the various participants.

In summary, the development control process is sufficiently comprehensive to permit the incorporation of storm water management concepts. Recognition and understanding of the concepts by planners, engineers, municipalities, and agencies involved in the development of land is now required.

6.4 Funding

The principles or the philosophy on which funding is based can vary widely depending on the interests and goals of the agency or program. The following is a summary of the grants and funding currently available from various agencies.

TABLE 48. STORM WATER MANAGEMENT CONSIDERATIONS IN THE PLANNING PROCESS

	Storm Water Management Aspects	Municipal Role	Provincial Agencies* Role	Developer's Role
1) Provincial Policy	- quantity and quality control.		- statement of specific criteria and standards to facilitate implementation of policy.	
	- statement of goals and objectives to be achieved in new development and existing problem areas.			
2) Official Plan	- statement of principles for storm water management.	- perform necessary background studies.	- provide technical assistance.	
	- identification of areas or watershed requiring special consideration.	- prepare plan.	- review and advise on draft plan to Ministry of Housing or regional municipality where applicable.	
	- statement of responsibilities in storm water management (watershed plan, design, approval, construction).			

*e.g., Ministry of Environment, Ministry of Natural Resources, Conservation Authorities, Ministry of Housing, etc.

TABLE 48. (CONT'D)

	Storm Water Management Aspects	Municipal Role	Provincial Agencies Role	Developer's Role
3) Subdivision Approval	- plan to be based on storm water management principles	- identify specific criteria and unusual constraints.	- provide specific criteria or identify unusual constraints.	- prepare draft plan of subdivision based on storm water management principles.
	- evaluation of the impact of the change in quantity and quality of drainage caused by the change in land use.	- give technical direction.	- give technical advice.	- provide evaluation of potential impact of the land use change.
	- storm water management plan which provides system to mitigate impact.	- identify special problems or needs for area under consideration.	- review and provide comment to approving authority.	- prepare storm water management plan.
	- erosion control plan for site protection during and after construction.			- prepare erosion control plan.
4) Subdivision Agreement	- specify action and responsibility on part of developer and municipality.	- full party to agreement.	- through conditions of draft plan of subdivision require certain matters be included.	- full party to agreement.
	- standards.			
	- monitoring (during and post-construction).			

TABLE 48. (CONT'D)

	Storm Water Management Aspects	Municipal Role	Provincial Agencies Role	Developer's Role
	- design. - construction. - ownership. - operation. - maintenance.			
5) Municipal Bylaw	- to state municipal requirements or standards for storm water management and erosion control to be adhered to. - provisions for certain areas. - provisions for certain activities. - review and approval process in municipality.	- prepared by municipality. - approved by municipal council.	- approval by Ontario Municipal Board.	
6) Municipal Engineering Standards	- state specific standard and design criteria for storm water management system and individual components. - must be responsive to advancements in the field. - must be flexible for unusual conditions.	- prepared and maintained by municipal engineering (works) department.		

TABLE 48. (CONT'D)

	Storm Water Management Aspects	Municipal Role	Provincial Agencies Role	Developer's Role
7) Ministry of Environment approval (Section 42 of Ontario Water Resources Act).	- under Section 42, OWR Act the plans, specifications and engineer's report must be submitted and approval obtained. - the report should document urban drainage conditions, effectiveness of managing the storm water and the potential problems.	- ensure that proposal meets higher of municipal standards or MOE standards.	- MOE reviews technical aspects and ensures that proposal meets environmental criteria.	- normally developers prepares submission on behalf of municipality.
8) Ministry of Natural Resources approval (Lakes and Rivers Improvement Act)	- where a natural watercourse is being altered or diverted.	- ensure that proposal meets higher of municipal standards or MNR standards.	- MNR reviews technical aspects and ensures that proposal meets criteria	- prepares submission.
9) Conservation Authority Approval	- under Section 27 of The Conservation Authorities Act. - control of activities in rivers, lakes and valleys.	- ensure that proposal meets higher of municipal standards or Authority standards.	- MNR approves regulations and criteria	- prepares submission.

6.4.1 Ministry of the Environment (MOE)

The Ministry of the Environment in Ontario administers the municipal infrastructure part of the Community Services Contribution Program. This program is described under Section 6.4.5. The Ministry also assists municipal water and sewage projects directly; however, storm sewage works are presently not eligible for funding under the provincial program.

6.4.2 Ministry of Agriculture and Food

Assistance in the amount of one-third of the cost for municipalities in a county and two-thirds of the cost for municipalities in a territorial district is granted for drainage works for agricultural lands.

6.4.3 Ministry of Natural Resources (MNR)

A grant for the construction of dams and reservoirs is available to Conservation Authorities on the basis of 50% of land costs and 50% of the first $30 000 of construction costs, and 100% of the costs over $30 000. The dam or reservoir must be for the prevention of flooding and the conservation of water.

The installation of other water control structures, such as works related to channel improvement, erosion control, etc., is assisted by the application of a 50% subsidy on the gross capital cost, or otherwise as approved by the MNR.

6.4.4 Ministry of Transportation and Communications (MTC)

Subsidies are provided by MTC to municipalities for the provision and maintenance of storm sewer facilities in conjunction with road works subject to certain limitations. For example, subsidies are not provided for the installation of storm sewers, catch basins, etc., in subdivision streets. Also, when a storm sewer greater than 69 cm (27 in) in diameter is installed, only that fraction of the cost equal to the ratio of 69 cm (27 in) to the total diameter is eligible. Subsidy for the storm sewer works is at the same rate as that applied to the road items in the project.

6.4.5 Canada Mortgage and Housing Corporation (CMHC)

An agreement between the Province of Ontario and the Federal Government of Canada dated March, 1979, established the Community Services Contribution Program (CSCP). The program consolidates funding previously provided under the Neighbourhood Improvement Program, the CMHC Municipal Infrastructure Program and the Municipal Incentive Grant Program.

The Ontario Ministry of the Environment administers the municipal infrastructure part of the program. Eligible projects are similar to the projects previously funded by the federal government under Part VIII of The National Housing Act of Canada and administered by CMHC. Storm sewer projects are eligible where the project provides for the collection and transmission of storm drainage that will open up land for residential development in previously undeveloped areas. The sewer downstream from the last connection to the collection system within the new subdivision is eligible. A storm sewer that serves existing development as well as opening up land for residential development is eligible for that portion that opens up land for residential development. Normally assistance is given for only a portion of the total cost of an eligible project. Items that qualify for assistance include construction costs, land costs, carrying costs and design, legal, and interim financing costs. Also eligible is the cost of land for flow-through ponds for reservoirs or ponds which are a component of an unconventional storm water collection system.

The Community Services Contribution Program provides for a grant of up to one-sixth of the eligible project costs for storm sewer projects.

A CSCP grant is also available to fund up to one-half of the study cost of a "regional sewerage plan" to establish an overall view on a multi-municipal or regional government level for storm drainage and sanitary sewage facilities. Local storm water management studies within a region may be eligible for subsidies if it can be demonstrated that such a study is an integral part of the regional plan.

6.4.6 Summary

Since water and sewage works receive higher priority than storm water facilities in the allocation of the MOE capital construction budget, it is likely that funds will be very limited for storm water

facilities for some time. The grants provided by CMHC are the most advantageous for the development of new lands. However, it appears that these grants are based upon a principle of piped sewer systems. Whether many of the storm water management measures discussed in this report would qualify for grants from CMHC is unclear. If the funding philosophy and criteria were altered so that they were compatible with the storm water management concept, their implementation would be more readily achieved.